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UPLIFT AND COMPRESSION BEHAVIOUR OF DRILLED PIERS IN LEDA CLAY

Report No 86-165-K

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ABSTRACT

The construction and load carrying capacity of augered footings in soft clay was investigated by a field program on full scale footings. Methods of construction and resistance to uplift and compression loading are described. The effect of factors such as long term load, cyclic load, combined load and age of footing on load resistance is discussed. Side adhesion and end bearing factors are recommended for design.

job	file	date	report no.
740643-520-433	843.2	September 17, 1986	86-165-K

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INTRODUCTION

The proposed 500 kV transmission lines in eastern Ontario, namely, Lennox GS to Hawthorne TS, Hawthorne TS to Merivale TS and Merivale TS to Cornwall TS traverse through areas covered with deep deposits of soft and sensitive clay. This clay deposit, commonly referred to as Leda clay, was formed in the Champlain Sea during a single marine episode that ended as a result of differential uplift of the land. Subsequent sediments were deposited first in esturine, and then in a fresh water, lacustrine environment/1,2/.

These clays, due to their unstable internal fabric or structure, exhibit a high sensitivity or loss of strength by disturbance and remoulding. Any physical disturbance of the clay structure causes the water to be released from the pores of the open fabric, as contact bonds are broken down and, in many instances, the entire mass becomes a viscous fluid.

Three features that characterize the unusual behaviour of Leda clay as it is found today are (1) the possibility of excessive settlements due to the open nature of the soil fabric, (2) the possibility of substantial loss of strength resulting from the breakdown of the soil structure, and (3) the possibility that the lost strength cannot be recovered through thixotropic processes due to the breakdown of the structure.

A wealth of information exists on the engineering behaviour of Leda clay deposits in the Ottawa valley. However, most of this information pertains to the behaviour of slopes and settlement of structures founded on this clay. Not much experience nor information exists in the design and construction of augered foundations supporting large loads such as those needed for 500 kV transmission towers.

Because of the low strength and high sensitivity of these clay deposits, the selection and design of appropriate types of foundations for transmission towers became an important consideration. In particular, the constructability and performance of augered concrete pier foundations under uplift, compression and overturning moment loads were examined. In consultation with the Stations & Transmission Design Department, a program of field testing of near full scale foundations was carried out at the former HMCS Gloucester Naval Base, south of Ottawa. The National Research Council has carried out research studies at this site where they constructed a test fill and monitored the settlement behaviour of the Leda clay deposits. This site was also used for evaluating several in situ soil test methods in cooperation with the Building Research Division of the National Research Council/3/.

This report summarizes the results of uplift and compression tests and discusses their implications on the design and construction of foundations for lattice type towers. The results of lateral load and overturning moment tests will be discussed in a separate report.

TEST PROGRAM

The test program was designed to address the following questions:

1. Constructability of large diameter (up to 2.7 m) and deep (up to 13 m) augered pier foundations in a soft Leda clay deposit.

2. Ultimate load and load-deformation behaviour of augered pier foundations in Leda clay under the type of loads induced by 500 kV lattice type transmission towers.

A total of 8 test footings, having a nominal size of 1.53 m in diameter and 6.4 m in depth, were installed for studying the following aspects of footing performance:

1. Ultimate load and load-deformation behaviour under short-term, uplift loads.

2. Effect of cyclic loading on uplift capacity.

3. Effect of construction methods on uplift capacity.

4. Thixotropic gain or development of the bond between concrete footing and the clay with time.

5. Effect of rate of loading on the uplift capacity.

6. Ultimate load and load-deformation behaviour under short-term compression loads.

7. Load-deformation behaviour under a ground line shear force.

The layout of the footings is shown in Figure 1. Footings 1, 3, 4, 5, 9, 10 and 11 were used for uplift testing. Footing 2 was tested in compression with Footings 1 and 3 acting as reaction supports. Footing 3 was tested under combined uplift and ground line shear loads. Footing 4 was tested under sustained or long-term uplift loading, whereas Footing 11 was tested under cyclic uplift loads. Table I gives a summary of the test footing dimensions and the types of loading applied.

SOIL PROPERTIES

The test site is located on marine deposits of the ancient Champlain Sea, which invaded the area following the retreat of the last continental ice sheet/2 /. For a time, the ice sheet formed the northern shoreline a few miles to the east of this site. Melt waters from the glacier transported and deposited clayey silts and sands into the sea. As the glacier retreated, fresh water streams transported clays and silts eroded from exposed soils farther inland; the inflow of fresh water diluted the sea and the soils were deposited in brackish water. As a result of these events and owing to underwater currents and possible ocean storms, the sediments now at the site are remnants of various stages of deposition, erosion and redeposition cycles.

A detailed characterization of the soil properties was made by means of undisturbed sample borings and other in situ tests such as vane shear, dilatometer, piezocone and pressuremeter tests/3/. The shear strength data are summarized in Figure 2 and are shown in more detail in Appendix A. The soil deposit over the test area was relatively uniform. Beneath the topsoil, a thin layer (about 1.5 m thick) of compact sandy silt was encountered below which extended the soft clay deposit with a very thin crust containing some decayed roots and wood material. The underlying gray silty clay extended to a depth of 18 m and contained 4 distinct zones that were identified by water content and Atterberg limit profiles. However, they were more clearly defined by the variations with depth of undrained shear strength and preconsolidation pressure (Figure 2). Undrained shear strength values varied between 18 and 30 kPa over the depth of the test footings (6.4 to 10 m). The surficial sandy silt layer was considerably stronger than the underlying clay deposit. The ground water table was near the ground surface and the site was fairly flat with poor drainage. The ground water table was monitored over the period from June to November, 1985 when the footing construction and testing was in progress.

TEST FOUNDATIONS

Size

Individual footing dimensions have been summarized in Table I. Of the eight augered footings tested in uplift and compression, seven were the same general size; 1.5 m diameter by 6.6 m deep, while uplift Footing 1 was 0.9 m diameter by 9.6 m deep. Using steel liners for excavation support during construction caused the upper portion of these footings to have a 4 to 20 cm larger diameter than the lower uncased portion. Enlarged diameters extended to depths ranging from 1.5 to 4.0 m below grade. The compression and uplift footings were cast with 30 to 60 cm projections

above grade. Flat recesses were formed in these projections for lateral load testing.

Materials

All footings incorporated a conventional cage of reinforcing steel. Concrete for the compression and uplift footings was specified to have a 28 day, unconfined compressive strength of 20 MPa. Pea sized gravel and a slump of 125 mm to 150 mm were employed to facilitate placement with a pumper truck and to ensure good contact with the many embedded instruments. Quality control tests on the concrete showed (Table II) that the specified compressive strength was achieved.

INSTRUMENTATION

Instruments were cast in the footings and placed in the soil next to the footings to measure the footing and the soil behaviour during the load tests. In particular, devices were installed to measure stresses and strains in the concrete footing, stress transfer at the soil/concrete interface, deformation of the soil and concrete, and pore pressure build-up in the soil. The instruments used are described below while their locations in each footing are detailed in the drawings in Appendix B.

Instruments in the Footings

Plastic pipes, extending from the top to the bottom of each footing, were cast in-place to provide access to the soil under the base of the footing. To prevent concrete entry, these pipes were pushed some 10 to 15 cm into the soil at the base of the excavations. These access tubes allowed piezometers to be installed in the base soil after foundation construction. They also allowed disturbance to the soil to be measured by soil tests performed either after construction or after load testing. As shown in Table III, which summarizes the instruments in each footing, all footings had at least one access tube and some had as many as three.

One vibrating wire (IRAD) piezometer was installed below each footing via the access tube. The pore pressure sensors were located 0.3 to 0.6 m below the base of the footings. Care was taken to prevent the piezometers from moving when the footings moved under load testing.

Three Glotzel pressure cells were installed on the base of compression Footing 2 (Figure 3) to separate end bearing and side friction capacity contributions. Glotzel and Petur (pneumatic) cells were placed on the sides of Footings 1 and 3 to monitor lateral earth pressure during lateral (shear) loading tests. These pressure cells were mounted on small jacks attached to the reinforcing cages (Figure 4) so they could be jacked against the sides of the footing excavation prior to concrete placing.

Strain gauged "sister bars" were attached to the reinforcing cages for Footings 1 and 2 to measure stress distributions in the footings. A load cell, consisting of several short strain gauged bars (Figure 5), was installed in the top of compression Footing 2. A measure of field cured concrete modulus was obtained from these load cells.

As shown in Table III, inclinometer casings were installed in Footings 1, 2 and 3 to measure deflection during lateral load tests. A portable tiltmeter, survey equipment, LVDT's, dial gauges and instrumentation in the soil supplemented the instrumentation cast in the footings. Only one instrument, a pneumatic pressure cell in Footing 1, was lost during construction. All other instruments performed satisfactorily.

Instruments in the Soil

Aside from the instruments cast into the concrete footings, a number of settlement gauges, piezometers, pressure cells and inclinometers were installed to monitor the soil behaviour during the loading tests on Footings 1, 2 and 3. An inclinometer was placed beside Footing 6 to determine if lateral soil deformation occurred during augering. For reference, several piezometers and a deep benchmark were installed at the edge of the site, away from the test footings. Table III summarizes the instrumentation placed in the soil. Instrument locations are detailed in Appendix B.

Calibration and Monitoring

The calibration of each instrument was checked in the laboratory prior to installing the device in the field. Readings were taken on the pressure cells during concrete placement to see if they were correctly sensing the fluid pressure of the overlying concrete. The functioning of all gauges was checked again a few days after construction was completed. Foundation testing did not take place for several weeks and, for some footings, months after construction. Initial readings were taken on all gauges prior to load testing and at regular load increments during the loading procedure.

CONSTRUCTION OF TEST FOOTINGS

The test footing program provided a significant opportunity to gain experience in constructing augered foundations in Leda clay. This section documents the site preparation that was done, the construction methods that were used, the response of the soil to these methods and the relevance of this experience to the future line construction.

Site Preparation

Initially the site was flat, poorly drained and overgrown with bushes. Construction personnel cleared the vegetation, laid a geotextile over the natural soil and spread a 0.30 m thick working pad of gravel mixed sand fill over the 60 m by 60 m test site. Horizontal and vertical survey control was established to permit accurate layout and construction of the test footings.

The working pad provided excellent support for the heavy construction equipment. It should be noted that the weather at the site was exceptionally dry during the construction periods in July and September. A thicker working pad or different fill material may be required to produce similar good traffic conditions in poor weather.

Construction Methods

Due to the tendency for Leda clay to collapse into a liquid when disturbed, considerable uncertainty surrounded the choice of an appropriate construction method. On a previous 230 kV transmission line near Arnprior, conventional foundation augering machines had liquified the natural clay within the footing dimensions and the clay slurry had been displaced by tremie concrete techniques/4/. Construction tests in similar soils in Norway had been successful in removing the native clay and maintaining the excavation open with a bentonite slurry, water with fly ash or water alone. Bentonite was eliminated from use when it was found to prevent the development of adhesion between the footing concrete and the surrounding native clay/5/. Several methods of excavating the clay, supporting the hole and placing the concrete were used on the test footings, as described below.

Excavation of the holes was accomplished with a Watson 3000 caisson drill fitted with an auger or a bailing bucket. A short steel liner was inserted in the top 3 m of the holes to prevent caving of the upper silty sand layer. The liner was usually at least 0.10 m larger than the auger, resulting in footings that had a larger diameter upper section and a smaller diameter lower section.

Excavation

Before constructing the test footings, a trial hole, 0.9 m in diameter, was augered at the southwest corner of the clear area. To minimize disturbance to the clay, the augers were corkscrewed into the ground 0.3 m to 0.6 m, then rotated a few turns without advancing to shear the clay on the auger from the surrounding soil. While this approach succeeded in keeping the bulk of the clay cuttings intact, a thin film of liquified, remoulded clay was formed on top of the auger which caused the cuttings to slip off. This problem was most pronounced with smaller diameter augers that have a steeper pitch in their flights. With larger diameter augers that have shallower pitched flights, the spoil still slipped off the augers but a greater depth of hole was accomplished before this occurred. By welding several large cleats on the top of the auger flights (Figure 6), slipping was halted and the cuttings were successfully removed from the holes. This method was used for all construction except the first test hole and the first footing (Footing 5).

As the use of cleats was not adopted until the second footing of the construction program (Footing 4), the remainder of the trial hole was advanced by remoulding the clay and sucking the resulting slurry from the hole with a septic tank cleanout pump. Clay remoulding was accomplished by slowly advancing the auger while it was being rotated quickly. Dilution of the slurry with a substantial volume of water was necessary before the pump would operate properly. Large lumps in the diluted slurry still caused occasional brief work stoppages. The water was supplied to the site and the waste slurry removed from the site in the septic tank cleanout truck. Slurry or water was removed from the top of the footing excavation at the same time and rate as the concrete was "tremied" to the bottom of the excavation.

A 0.75 m diameter bailer bucket with a bottom trap door was used to remove the cuttings from Footing 5 that could not be removed by the 1.5 m auger (without cleats). The bucket successfully removed the remoulded clay from the hole but was slow because it was small compared to the hole diameter. An attempt to excavate with this bucket appeared to be unsuccessful as the suction produced by withdrawing the bucket caused the soft clay to collapse into the hole.

Excavation Support

Due to the low soil strength, excavations in soft clay are susceptible to side failure or bottom heave. The length of time an unsupported auger hole would remain open in this material was believed to be very short. While long liners could have been used, they are difficult to handle and install plumb and were not compatible with the desired test footing instrumentation. Bentonite slurry is frequently used to prevent hole collapse because it is readily displaced by concrete and does not adhere to the reinforcing steel. It was not used in this program due to the reported bad experience with its use in Norwegian clays.

Only the first test auger hole was supported with a clay slurry created by remoulding the native soil. This approach made inspection of the augered hole impossible. Visual examination of a reinforcing steel rod placed in the test hole during concreting left doubts whether the concrete would completely remove this clay from the steel in order to form a good concrete to steel bond.

Footings 4 and 5 exhibited some disturbance at the bottom of the augered excavation. While this may have been due to cuttings which could not be cleaned from the hole or due to the suction created when withdrawing the auger, it was interpreted as incipient bottom heave. These excavations were filled with 4.25 m of water to stabilize the base of the holes. Water was effective in keeping these holes open. Concrete was tremied to the bottom of these footings to complete footing construction.

No support, apart from the short liner through the surface sand, was necessary for any of the other footings. Virtually all these footings exhibited a characteristic concentric ring pattern from the auger teeth indicating no disturbance at the base of the excavation (Figure 7). The footings remained stable and open until concrete was placed, which was usually 1/2 to 1 hour after excavation was completed. A 2.76 m diameter by 12.7 m deep footing boring, excavated for moment testing, remained open for over 4 hours without significant deterioration.

In clay deposits similar to this site, normal open hole construction procedures should be successful. Quick placement of concrete, after excavation, is recommended to limit the risk of hole collapse. Where these clay deposits contain wet silt layers, shorter stand-open time may be expected and a change in construction method might be required.

Concrete Placing

Two methods of concrete placing were employed in the test footing construction: a conventional gravity tremie pipe and a pumper truck. Both methods worked well. Most footings were "poured" using the pumper truck because its smaller pipe was able to fit through the instrumentation mounted on the reinforcing cages. The tremie pipe was too large for this purpose, so it was only used on Footing 4.

Forces produced by the high velocity of concrete exiting the pump truck pipe were expected to produce enlarged bases on the test footings. While this might be desirable for production footings it was extremely undesirable when concrete was pumped for the test footings. An initial cubic metre was placed slowly before the fast pumping rate was used, so a cushion was formed to protect the weak soil beneath the test footings. Records were kept of the concrete volume poured in each footing and compared to the excavation volume. Extremely close agreement between these quantities suggest that no basal bulbs were formed by pump truck construction of these footings. A 1 to 2 m^3 discrepancy occurred in the test footing, when the initial cushion volume technique was not used, indicating a basal bulb had formed.

Either the gravity tremie pipe or pump truck is suitable for production footing construction. When a pump truck is used, the initial concrete volume should be placed slowly to avoid failing the soil at the base of the footing.

Soil Disturbance

Several aspects of the construction process cause soil disturbance which may decrease foundation capacity. Disturbance to the sides and base of the excavation was assessed qualitatively. The augering method employed produced suction forces on the soil when the auger was repeatedly pulled from the hole. This moderately disturbed the base of the hole in three of eleven footings. For the remaining footings, no significant disturbance occurred. Static cone tests, performed through the footings, showed no noticeable change in soil strength from before to after construction. However, these tests could not evaluate the 10 cm of soil immediately below the footing.

Due to the mucking out process, remoulded clay was smeared along the sides of the excavation as shown in Figure 8. Apart from this layer of remoulded soil, the sides of the hole were judged to be undisturbed. No significant lateral squeezing or caving of the holes was observed. An inclinometer was installed 1.5 m away from Footing 6 (2.4 m diameter) and monitored during the footings construction. No significant lateral soil movements were detected by the inclinometer.

Concrete placement by pumper truck has the potential to fail the soil. However, when this placing equipment was carefully controlled, disturbance to the soil was avoided.

Unloading of the soil by excavation and reloading by concrete placement has the potential to alter the soil strength through cyclic stressing. Laboratory stress path tests are in progress to duplicate this cycling and evaluate its effect on ultimate soil resistance.

LOAD TESTING

Six uplift tests, one compression test and two lateral shear tests were performed on seven of the footings. Testing of the eighth footing (Footing 5) was deferred to a future date. Table IV shows the sequence of testing, the type of tests performed on each footing and the specific technical question addressed by particular tests. It may be seen that the testing included:

- uplift on young age footing
- uplift with cycled loads
- uplift combined with lateral load
- uplift on a footing embedded only in the clay
- uplift loading applied slowly, to measure long term creep
- compression

It should be noted that Footings 1 and 3 provided the reaction for the compression test on Footing 2.

Schematics, descriptions and photographs of the various load tests are presented in Appendix C.

LOAD TEST RESULTS

Field Data

Table V summarizes foundation weights, contact areas with each soil layer and ultimate loads measured in the uplift or compression tests. Figures 9 to 15 show the load versus deflection curves for each test. A complete summary of the load test data is contained in Appendix C.

It may be seen from Table V that Footing 1, the smallest (0.9 m) diameter footing, had the lowest uplift capacity at 445 kN. This footing had a contact area with the clay similar to that of the larger (1.5 m) diameter footings but had less weight and less contact area with the gravel and silty sand layers (Table V).

Footing 10, the 1.5 m diameter footing tested with the surface soil layer removed, had an uplift capacity of 654 kN (Figure 12). For the remaining 1.5 m diamter footings uplift capacity ranged from 534 kN to 948 kN. The uplift tests, in order of increasing capacity were:

Long Term	Footing 4	534 kN	(Figure 14)
Combined Shear & Uplift	Footing 3	694 kN	(Figure 13)
Cyclic Load	Footing 11	828 kN	(Figure 12)
Youngest Age	Footing 9	948 kN	(Figure 10)

Up to 50% of these uplift capacities were contributed by the weight of the concrete footings. Table V shows footing weight contributed from 150 kN to 348 kN of the uplift resistance.

Despite the differences in loading procedures, all uplift tests behaved in a brittle manner. Load increased rapidly to near maximum value with little deflection below deflections of about 5 mm. All curves had a distinct knee in the vicinity of a 5 mm deflection. Past this point, load resistance does not increase appreciably with larger deflections. Figures 11 and 13 show this similarity in load-deflection response of Footings 9, 10 and 11.

The compression test, on Footing 2, measured an ultimate capacity of 595 kN. This maximum load was not achieved until the footing had settled more than 50 mm. While this capacity is similar to the uplift capacities, the soil is resisting both load and the weight of the footing in the compression case, while the weight of the footing aids the soil in resisting load in the uplift case. Figure 15 shows the contribution of the end bearing component to compression capacity, as measured by the base-mounted pressure cells. Instrument results are discussed in the next section.

Instrumentation Data

For the compression test, the three pressure cells at the base of the footing were monitored at every load increment. At the ultimate total load, the average pressure on these cells was 209 kPa. Significant load was carried by the base even at low deflections. Average pressure readings were used to calculate the end bearing component shown in Figure 15. Individual cell readings are summarized in Appendix D.

The piezometers located 0.6 m below the base of the uplift footings recorded decreases in pore water pressure (suction) at maximum load ranging from 28 kPa to 128 kPa. Figure 17 shows that the pore pressure change is very low at low deflections, typical of the working load range, but increases rapidly at higher deflections once the knee of the load deflection curve is passed. The pore pressure increased by 198 kPa below the compression footing during load testing. As in the uplift tests, the pore pressure change in the compression test was small at low deflections and increased rapidly at high deflections. Appendix D shows the details of pore pressure response with progressive increases in foundation load.

Pore pressures monitored beside the test footings did not change significantly. Only the settlement gauges nearest the footing (ie within 15 cm) recorded much movement. These results suggest the failure occurs quite close to the face of the footing.

Data Interpretation

Back analysis of the uplift tests was carried out to determine the adhesion acting between the surrounding clay and augered footings. In each case, the contributions of footing weight and friction from the upper granular layers were subtracted from the measured ultimate load to obtain the total resistance contributed by the clay. K_u theory was used to calculate the

side friction contributed by the gravel and sandy silt layers. These calculations used angle of friction values of 35° for the gravel and 31° for the sandy silt. As the contribution from end-suction is very small at low deflections, this factor was not deducted from the total resistance in determining the clay layer resistance. Unit adhesion values were obtained for each footing by dividing the total clay resistance by the surface area of the footing in contact with the clay layer.

Table V summarizes the results of these calculations. Resistance contributed by the surface granular layers ranged from 57 kN to 102 kN, except at Footing 10 where these layers were removed. The load carried by the clay layer ranged from 165 kN to 498 kN. Adhesion values, averaged over the footing surface embedded in clay, ranged from 8.9 kPa to 18.9 kPa.

The lowest adhesion value of 8.9 kPa was obtained in the long term loading test. This reduced adhesion was anticipated from similar behaviour in long term clay tests at Nanticoke/6/. In the other tests, the resistance of the clay does not appear to have been affected by the method of loading or the age of the footing. The clay resistance was not reduced by cyclic loading nor by combining lateral shear with the uplift load. The footing tested at the youngest age had the highest resistance, and as such, increased adhesion with time was not observed. Excluding the long term test, the average adhesion value was 14.7 kPa in uplift. Dividing this value by the average dilatometer shear strength of 22.5 kPa over the footing depth, yielded a value of 0.65 for the adhesion factor, α.

In the compression test, the back calculated values for the end bearing factor, N_c, and the adhesion factor, α, depended on how much of the footing weight was carried by end bearing and side adhesion, respectively. Just prior to the start of the compression test, an average pressure of 49 kPa was acting on the pressure cells at the foundation base. This initial base pressure corresponded very closely to the difference in unit weight between the concrete and the soil applied over the height of the footing or about one third of the footing weight. In the following discussion, it is assumed that side adhesion carried the other 2/3 of the foundation weight. The initial base pressure has been excluded from both the total load and the end bearing contribution shown in Figure 15. When this initial pressure is added to the end bearing pressures, generated by the loads applied during the compression, and the sum N_c value is divided by the 27 kPa clay shear strength at the footing base, the N_c value is about 6.73 at the knee of the load deflection curve and about 9.5 at the ultimate failure load (Figure 16). The Canadian Foundation Engineering Manual/7/ suggests that N_c be reduced from the theoretical value of 9 to 6 for footings exceeding 1.0 m in width. N_c values much greater than 6 are clearly mobilized in the compression test at Gloucester and an N_c value of 9 is recommended for tower foundation design.

When the load bearing contribution in Figure 16 is subtracted from the total applied load, the total shaft resistance for the compression footing, shown in Figure 17, is obtained. The shape of this curve is identical to the uplift test curves. Past the knee of this curve, side adhesion is essentially constant and has a unit value of 16.2 kPa. This is within the range of values obtained in the uplift tests. It is recommended that an

α factor of 0.65 be applied to the average in situ shear strength to determine the shaft resistance.

Design Considerations

Due to the low strength of the marine clay, the end bearing resistance is low and increases only gradually with depth. Foundation weight increases more rapidly with depth than end bearing or side resistance. Therefore, compression load design will be more critical than uplift for sizing tower foundations in this soft clay. Reducing the foundation weight, by pouring some of each footing with light weight aggregate concrete, or increasing the end area for bearing by belling are two possible approaches to solving this problem without sacrificing too much uplift capacity. A large diameter footing is anticipated for lateral load resistance so the choice of footing geometry will be limited. Perhaps these footings will have to be carried to the depth of competent soil underlying the clay to develop adequate compression capacity. If such a layer is not available within a technical or economically feasible depth, an alternative foundation to augered footings may have to be used.

SUMMARY AND CONCLUSIONS

The field work showed that heavy equipment could travel on a soft clay site with the help of a normal granular working pad. By welding small cleats on the top of a conventional auger, augered footings were excavated with conventional equipment. In pure clays, such as at this site, full scale footing excavations should remain open for a sufficient period of time to allow placing of the reinforcing cage and pouring of the concrete. Bentonite should not be used at sites where caving silts are layered with the soft clay. Only water should be used for supporting the excavation.

In the uplift test, the clay to concrete adhesion did not improve with aging. Adhesion was not adversely affected by cyclic loading or by combining the uplift with shear loading. Long term or sustained loading reduced the mobilized adhesion by a factor of 0.6. The adhesion operating in the compression test was comparable to that observed in the uplift tests. Calculated end bearing factors match well with theory. For design calculations of shaft adhesion, an adhesion factor α value of 0.65 is recommended for most loading situations. Where sustained loads occur, such as at angle towers, a reduced value of 0.40 is recommended. To determine end bearing, an N_c value of 9 should be used.

ACKNOWLEDGEMENTS

The successful completion of this field testing program is the result of a team effort and assistance by many departments of Ontario Hydro and outside agencies. The following are acknowledged for their valuable contributions:

1. The National Capital Commission and the Department of National Defence for permitting the field work to be carried out on their property at the Gloucester Naval Base.

2. The National Research Council for their technical cooperation in performing in situ soil property measurements and instrumentation of the test footings.

3. Mr. R. Tsang for his field coordination of the construction and testing activities and assistance in the anlaysis of test data.

4. Messrs. D. Fraser, M. Grills and P. Adams for carrying out installation and monitoring of instrumentation.

5. The Mechanical Testing Section of the Mechanical Research Department for the load testing.

6. The Lines and Stations Construction Department of Ontario Hydro for installing footings and assistance in load testing.

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REFERENCES

1. Gadd, N.R. "Surficial Geology of Ottawa Map-Area Ontario and Quebec 31 6/5". Geological Survey of Canada, Dept of Mines and Technical Surveys, Paper 62-16, 1963.

2. Karrow, P.F. "The Champlain Sea and its Sediments". Soils in Canada, University of Toronto Press, in Cooperation with the Royal Society of Canada, pp 97-108, 1961.

3. Bechai, M. "In Situ Testing Methods and their Application to Geotechnical Investigation for Transmission Lines. Research Division Report No 86-143-K.

4. Conversations with O.A. Johnston, Ontario Hydro.

5. Conversations with E. Di Baigio, NGI, 1985

6. Radhakrishna, H.S., and J.I. Adams. "Long Term Uplift Capacity of Augered Footings in Fissured Clay". Canadian Geotechnical Journal, Vol 10, No 4, 1973.

7. Canadian Foundation Engineering Manual, 1985.

TABLE I

Summary of Footing Dimensions as Constructed

Footing Number	Steel Liner: Embedded Length (m)	Steel Liner: Projection Above Gravel Fill (m)	Footing Diameter: Cased Portion (m)	Footing Diameter: Uncased[i] Augered Portion (m)	Measured Augered Length (m)	Projection of Footing Above Gravel Fill (m)	Type of Load Tests
1[+]	2.59	0.76	0.94	0.91	9.65	0.30	uplift
2	1.83	0.30	1.55	1.45	6.40	0.46	compression
3	1.98	0.15	1.55	1.45	6.40	0.30	uplift
4[+,*]	1.45	0.69	1.55	1.45	6.81	0.30	uplift
5[+,*]	1.52	0.61	1.55	1.45	6.32	0.30	uplift
6	2.92	1.35	2.53	2.44	6.63	3.35	moment
7	3.20	1.37	1.70	1.50	6.93	3.33	moment
8[!]	2.79	1.47	2.79	2.72	12.73	3.35	moment
9	4.04	0.53	1.70	1.50	6.88	0.61	uplift
10	3.96	0.61	1.70	1.50	6.71	0.61	uplift
11	3.71	0.86	1.70	1.50	6.65	0.61	uplift

Additional Notes: All of the footings constructed in the dry with exceptions as noted:
(+) noticed soil disturbance at bottom of augered hole
(*) water added to support hole (4.27 m deep)
(!) minor caving at side of hole below liner after 5 hours being left open and reinforcing cage sank to depth of 12.95 m
(i) auger diameter plus estimated "auger whip" of 2.5 to 5 cm

TABLE II

Summary of Concrete Properties of Laboratory Cured Cylinders

Footing No	Specified 28 day Compressive Strength (MPa)	Unconfined Compressive Strength (MPa)	Average Dry Density (kg/m^3)	Average Elastic Chord Modulus ($x10^3$ MPa)
1	20	23.6	2289	24.8
2	20	27.1	2331	28.7
3	20	20.6	2282	-
4	20	23.1	2298	27.3
5	20	22.1	2281	-
6	30	31.8	2300	31.0
7	30	31.6	2315	30.4
8	30	31.9	2317	32.4
9*	20	16.6	-	-
10	20	20.8	-	-
11	28	24.6	-	-

NOTE: (*) 14 day strength only; all other cylinders tested at 28 days or at time of foundations testing

TABLE III

Foundation Instrumentation*

A IN FOOTING

Footing No	Instrumentation Cast in Footing						
	Access Tubes	Base Piezometer	Strain Gauges	Load Cell	Pressure Cells Side	Pressure Cells Base	Inclinometer
1	1	✓	8	✓	4	-	✓
2	2	✓	6	✓	-	3	✓
3	1	✓	-	-	4	-	✓
4	2	✓	-	-	-	-	-
5	2	✓	-	-	-	-	-
9	3	✓	-	-	-	-	-
10	3	✓	-	-	-	-	-
11	3	✓	-	-	-	-	-

B BESIDE FOOTING

Footing No	Instruments in Soil			
	Piezometers (including base)	Settlement Gauges	Pressure Cells	Inclinometers
1	4	6	1	1
2	1	6	5	1
3	1	-	-	2

*see Appendix B for detailed location drawings of these instruments

TABLE IV

Foundation Testing Sequence

Test No	Footing No	Test Type	Test Details	Footing Age (weeks)
1	9	uplift	effect of age	2
2	11	uplift	cycled loads	5
3	2	compression		12
4	1	uplift	effect of scale	12
5	3	combined	lateral & uplift loads	12
6	2 & 3	lateral		12
7	10	uplift	sand & gravel layers removed	5
8	1 & 2	lateral	scale effect	13
9	4	uplift	long term loads	13
not tested	5	uplift		

TABLE V

SUMMARY OF UPLIFT TEST RESULTS

Footing No	Depth (m)	Diameter (m)	Total Length (m)	Weight Ftg (kN)	Contact Area (m^2) Gravel and Silty Sand	Contact Area (m^2) Clay	Ultimate Load (kN)	Ultimate Capacity Contributed From Gravel Pad and Silty Sand (kN)	Ultimate Capacity Contributed from Leda Clay (kN)	Average Adhesion of Leda Clay to Concrete (kPa)
uplift 1	0-2.59 2.59-9.73	0.9398 0.9144	10.03	150.50	4.95	23.20	444.9 (100 kips)	57.4 (13 kips)	237.0 (53 kips)	10.2 (213 psf)
lateral& uplift 3	0-1.98 1.98-6.48	1.5494 1.4478	6.78	263.32	8.16	21.93	694.2 (156 kips)	93.7 (21 kips)	337.1 (76 kips)	15.4 (321 psf)
long term 4	0-1.45 1.45-6.88	1.5494 1.4478	7.19	275.52	8.16	25.47	533.9 (130 kips)	93.7 (21 kips)	164.7 (47 kips)	8.9 (185 psf)
uplift 9	0-4.04 4.04-6.96	1.7018 1.4986	7.57	347.90	8.96	26.38	947.8 (213 kips)	101.9 (23 kips)	497.9 (112 kips)	18.9 (394 psf)
excavated sand 10	0-3.96 3.96-6.78	1.7018 1.4986	7.39	340.00	0.00	23.05	654.2 (147 kips)	0.00	314.2 (70 kips)	13.6 (285 psf)
cyclic 11	0-3.71 3.71-6.73	1.7018 1.4986	7.34	335.07	8.96	25.09	827.7 (185 kips)	101.9 (23 kips)	390.6 (88 kips)	15.6 (325 psf)

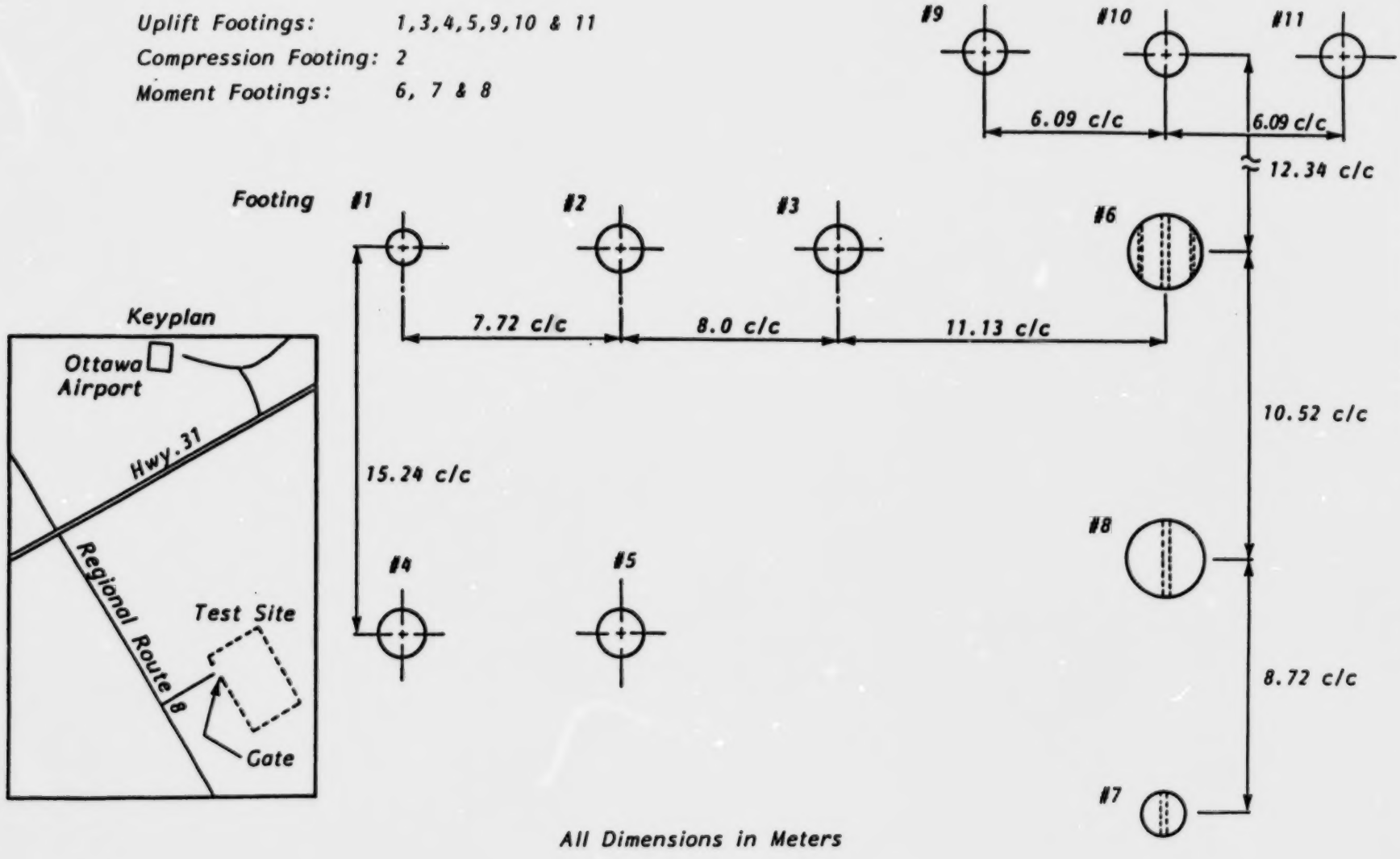


FIGURE 1

LAYOUT OF FOUNDATION FOOTINGS
AT GLOUCESTER TEST SITE

86-165
170516 RD

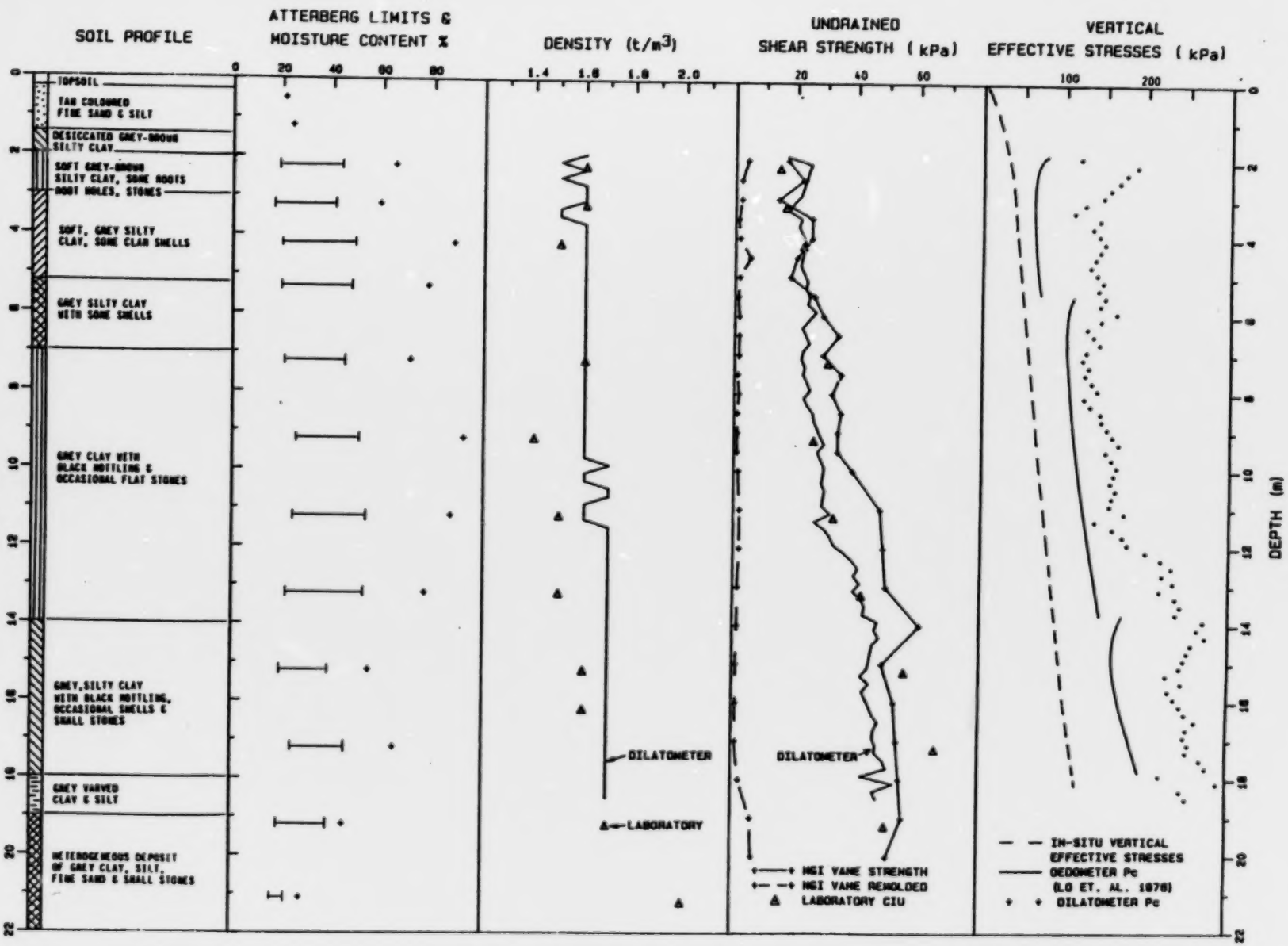

SOIL PROFILE
ATTERBERG LIMITS & MOISTURE CONTENT %
DENSITY (t/m3)
UNDRAINED SHEAR STRENGTH (kPa)
VERTICAL EFFECTIVE STRESSES (kPa)
TOPSOIL
TAN COLOURED FINE SAND & SILT
DESICCATED GREY-BROWN SILTY CLAY
SOFT GREY-BROWN SILTY CLAY, SOME ROOTS ROOT HOLES, STONES
SOFT, GREY SILTY CLAY, SOME CLAM SHELLS
GREY SILTY CLAY WITH SOME SHELLS
GREY CLAY WITH BLACK MOTTLING & OCCASIONAL FLAT STONES
GREY, SILTY CLAY WITH BLACK MOTTLING, OCCASIONAL SHELLS & SMALL STONES
GREY VARVED CLAY & SILT
HETEROGENEOUS DEPOSIT OF GREY CLAY, SILT, FINE SAND & SMALL STONES
DEPTH (m)
DILATOMETER
LABORATORY
NGI VANE STRENGTH
NGI VANE REMOLDED
LABORATORY CIU
IN-SITU VERTICAL EFFECTIVE STRESSES
OEDOMETER Pc (LO ET. AL. 1978)
DILATOMETER Pc

FIGURE 2

SOIL PROFILE FOR TEST SITE AT GLOUCESTER

86-165

171477 RD

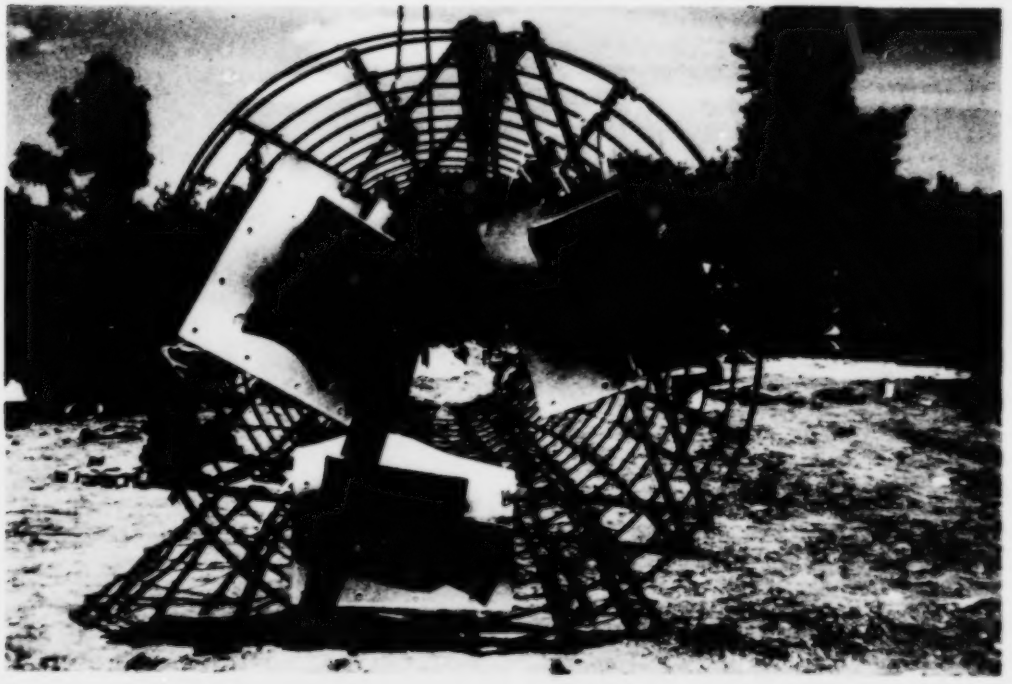


FIGURE 3

GLOTZEL PRESSURE CELLS FOR
COMPRESSION TEST FOOTING NO. 2
THE CELLS ARE MOUNTED ON REINFORCEMENT CAGE

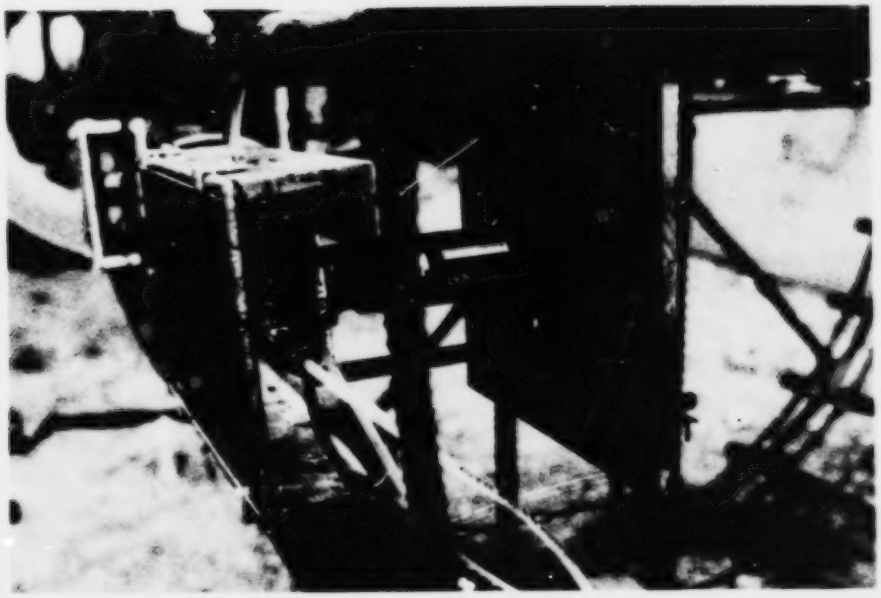


FIGURE 4a

LATERAL EARTH PRESSURE CELL MOUNTED ON A
SMALL JACK ATTACHED TO REINFORCING STEEL CAGE

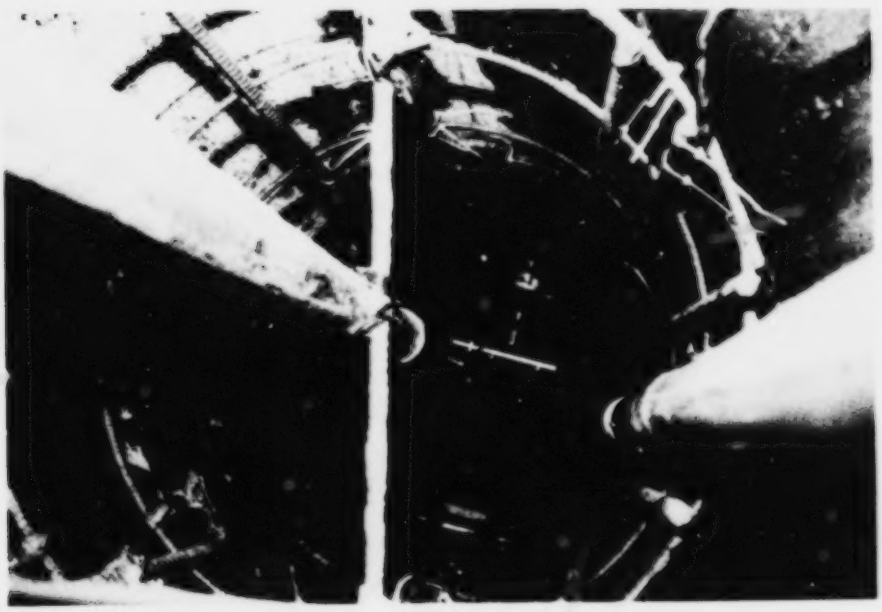


FIGURE 4b

LATERAL EARTH PRESSURE CELL PUSHED IN CONTACT
WITH CLAY BY MEANS OF A HYDRAULIC JACK
WHILE CONCRETE PUMPING PROGRESSES

FIGURE 5

LOAD CELL ASSEMBLY MADE OF STAIN GAUGED BARS
EMBEDDED IN COMPRESSION FOOTING 2

FIGURE 6

CLEATS WELDED ON THE AUGER FLIGHTS TO STOP THE EXCAVATED CLAY FROM SLIDING BACK IN TO THE AUGER HOLE

FIGURE 7
AUGER TEETH MARKS AT THE BOTTOM OF HOLE

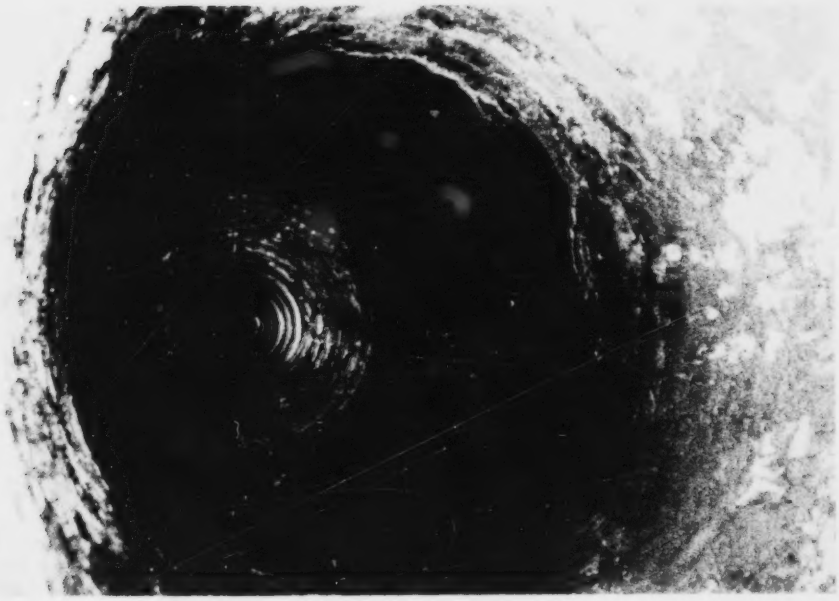


FIGURE 8
REMOULDED CLAY SMEAR ON THE SIDES OF HOLE

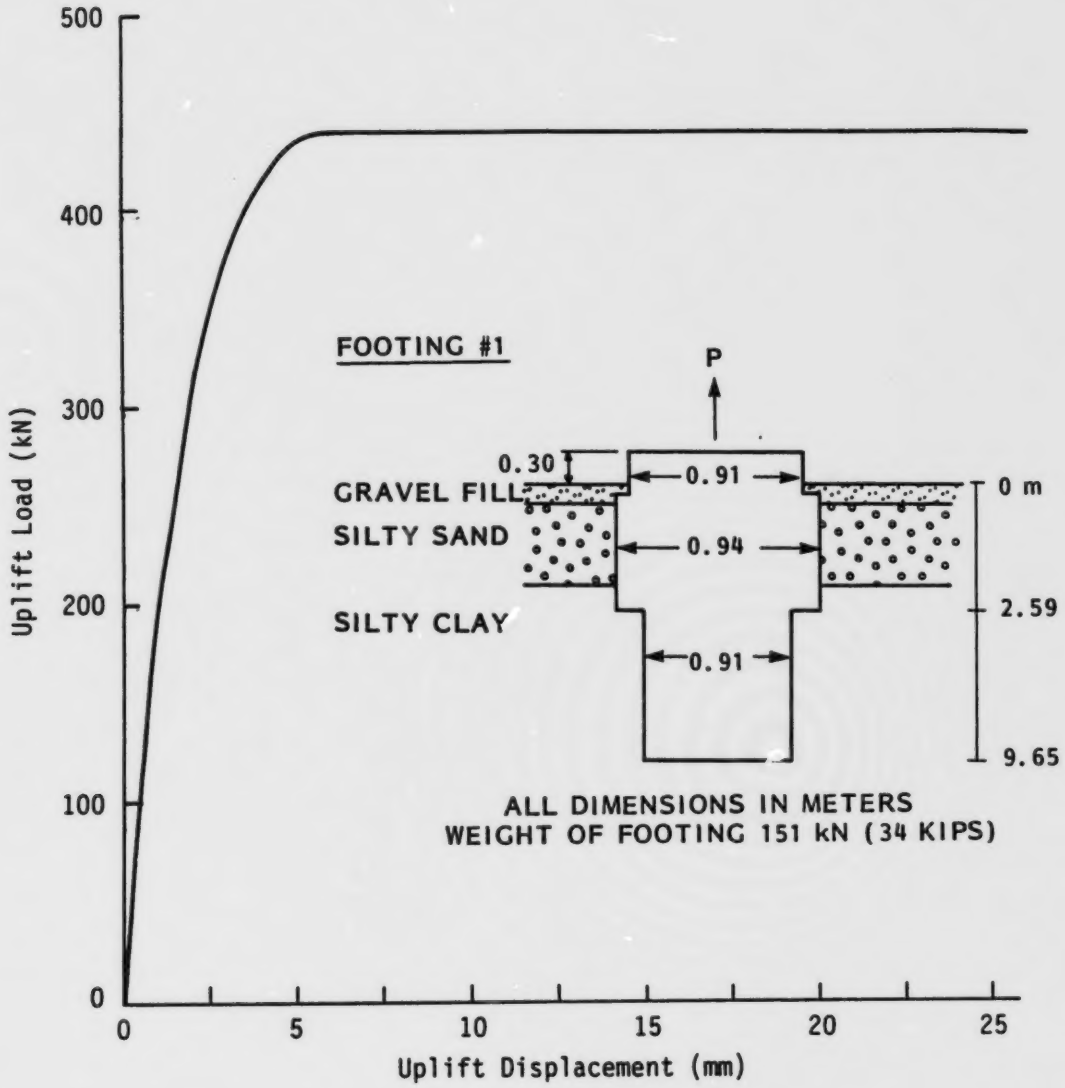


FIGURE 9

LOAD - DISPLACEMENT PLOT FROM UPLIFT TEST ON FOOTING #1

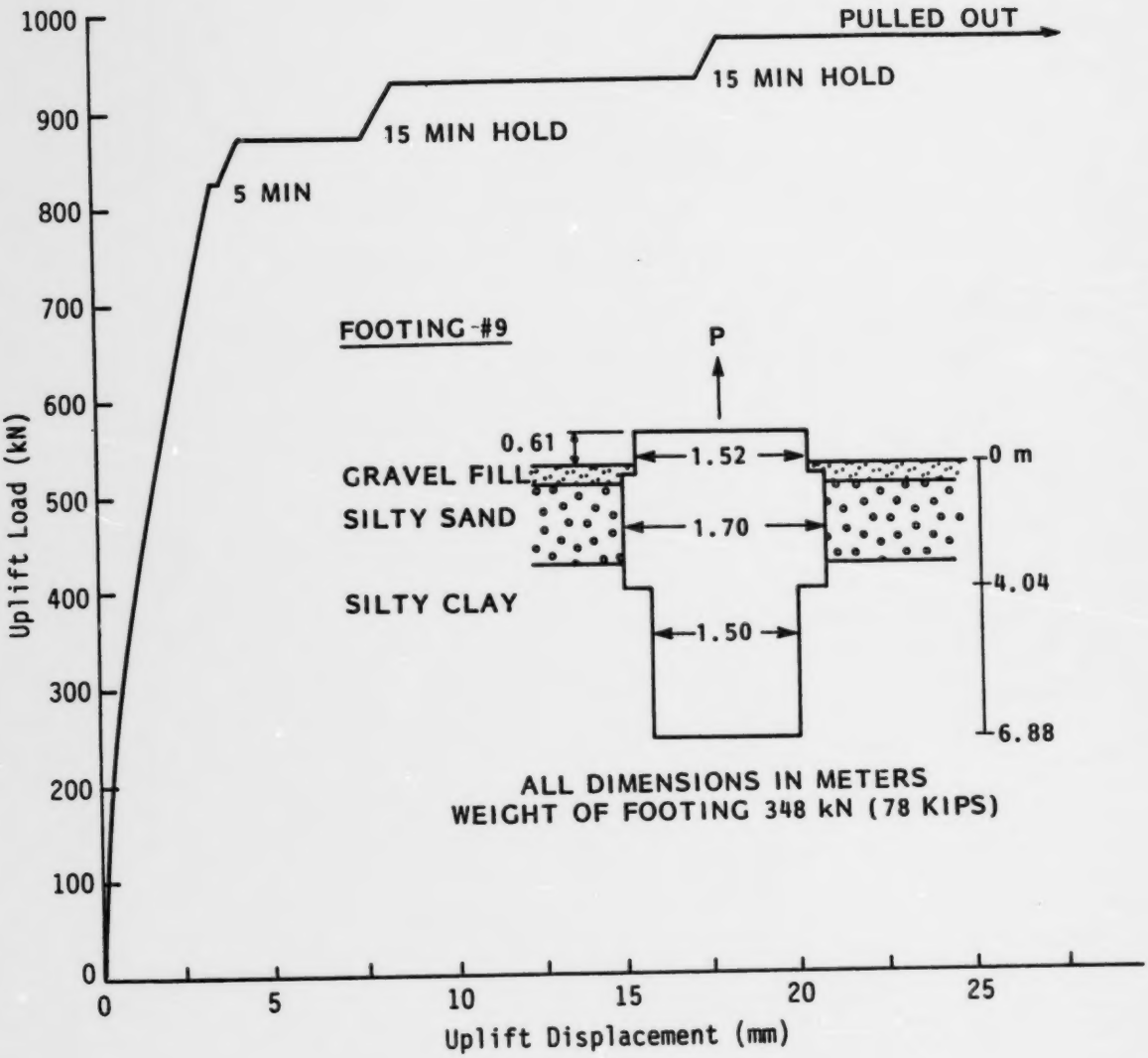


FIGURE 10

LOAD - DISPLACEMENT PLOT FROM UPLIFT TEST ON FOOTING #9

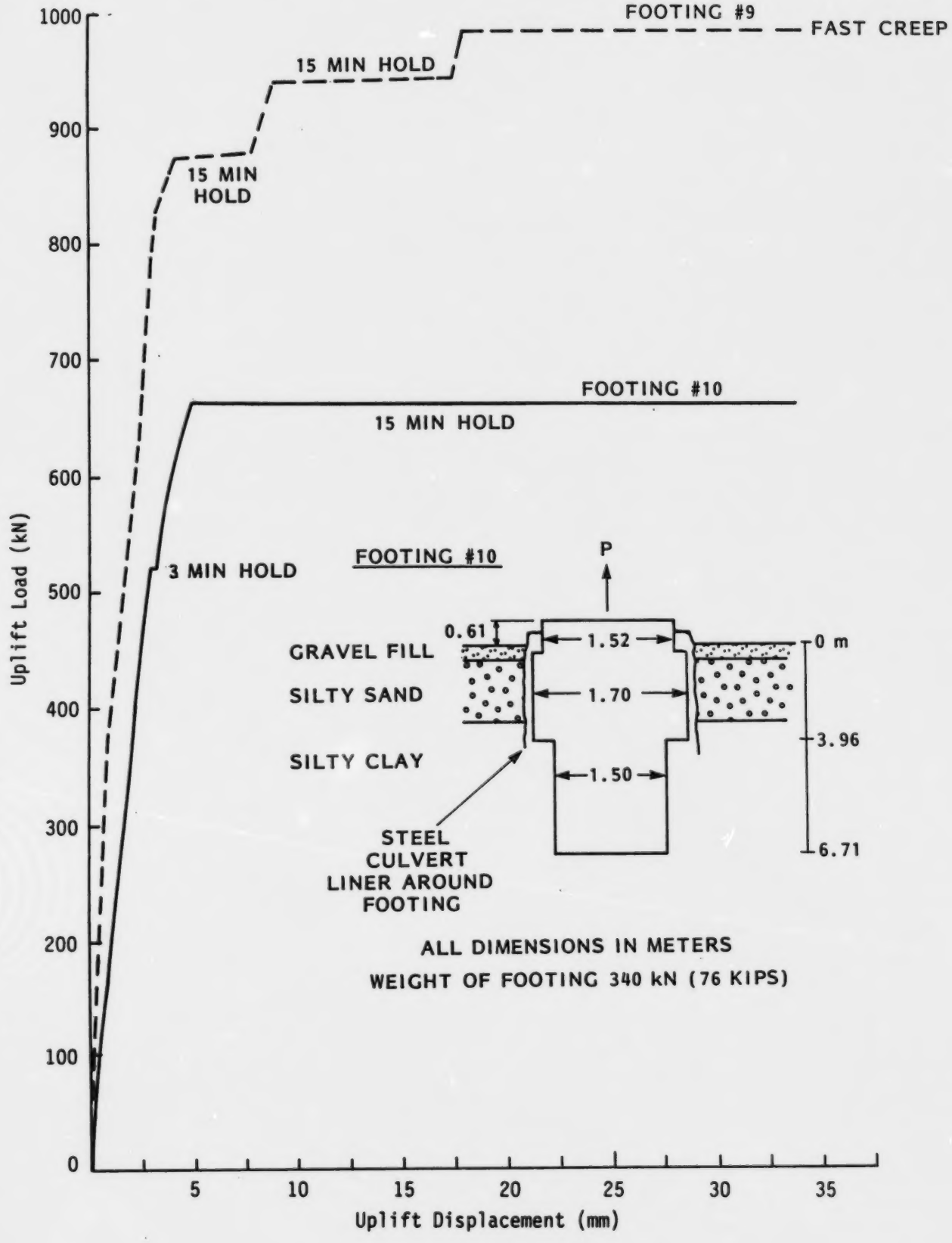


FIGURE 11

UPLIFT TEST WITH SILTY SAND EXCAVATED FROM FOOTING #10

86-165

171295 RD

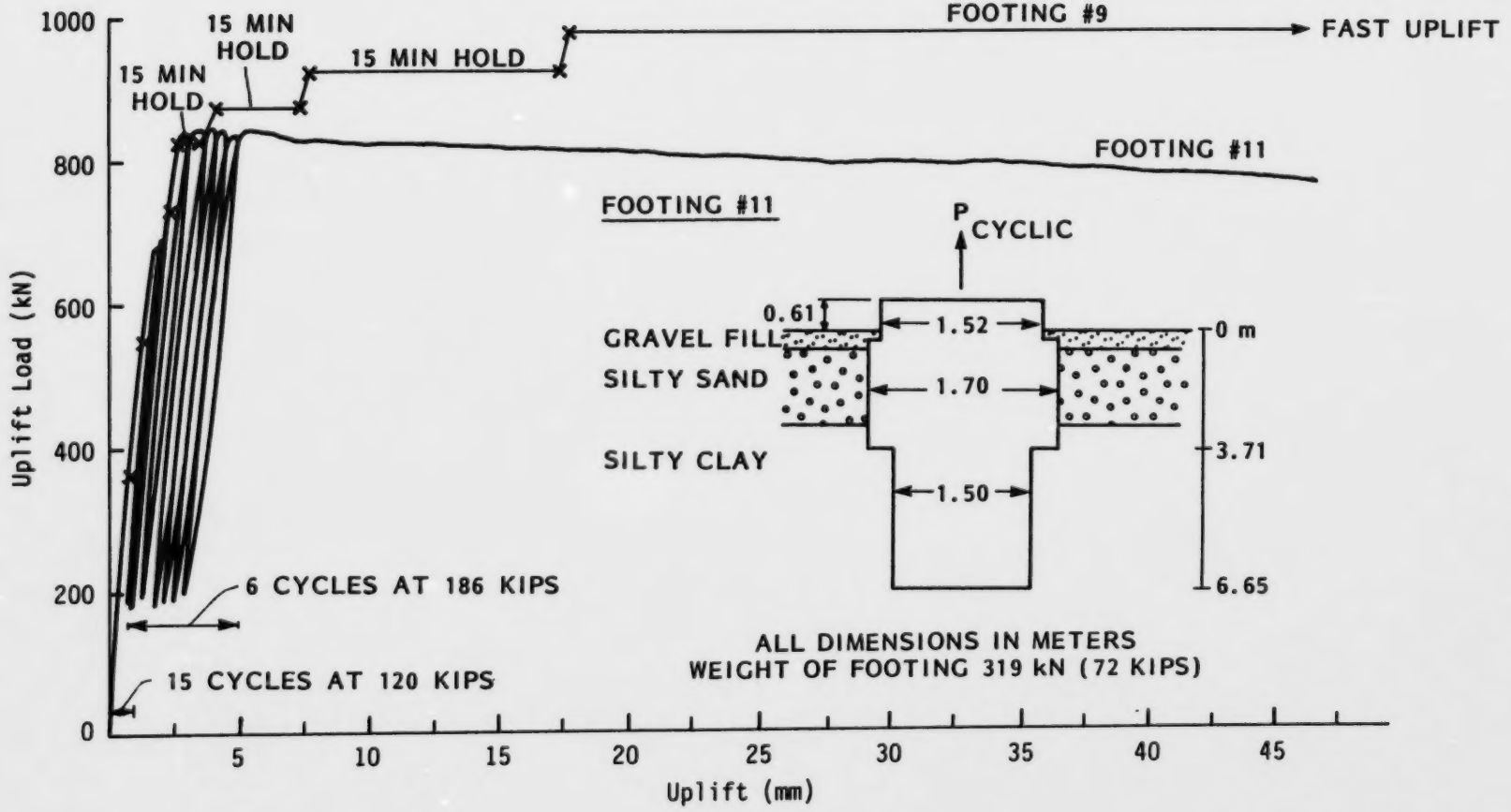


FIGURE 12

CYCLIC UPLIFT TEST ON FOOTING #11

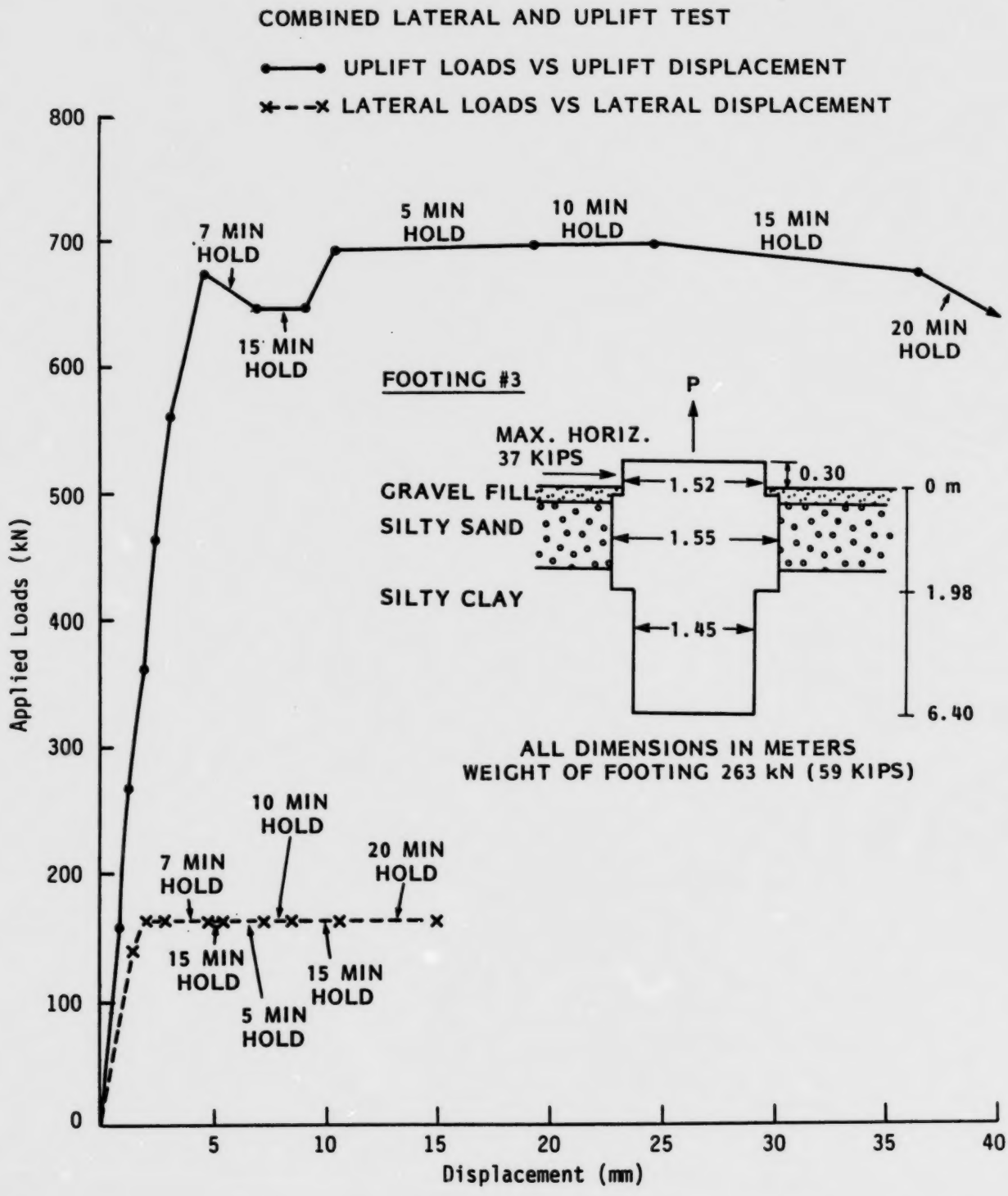


FIGURE 13

COMBINED LATERAL AND UPLIFT TEST ON FOOTING #3

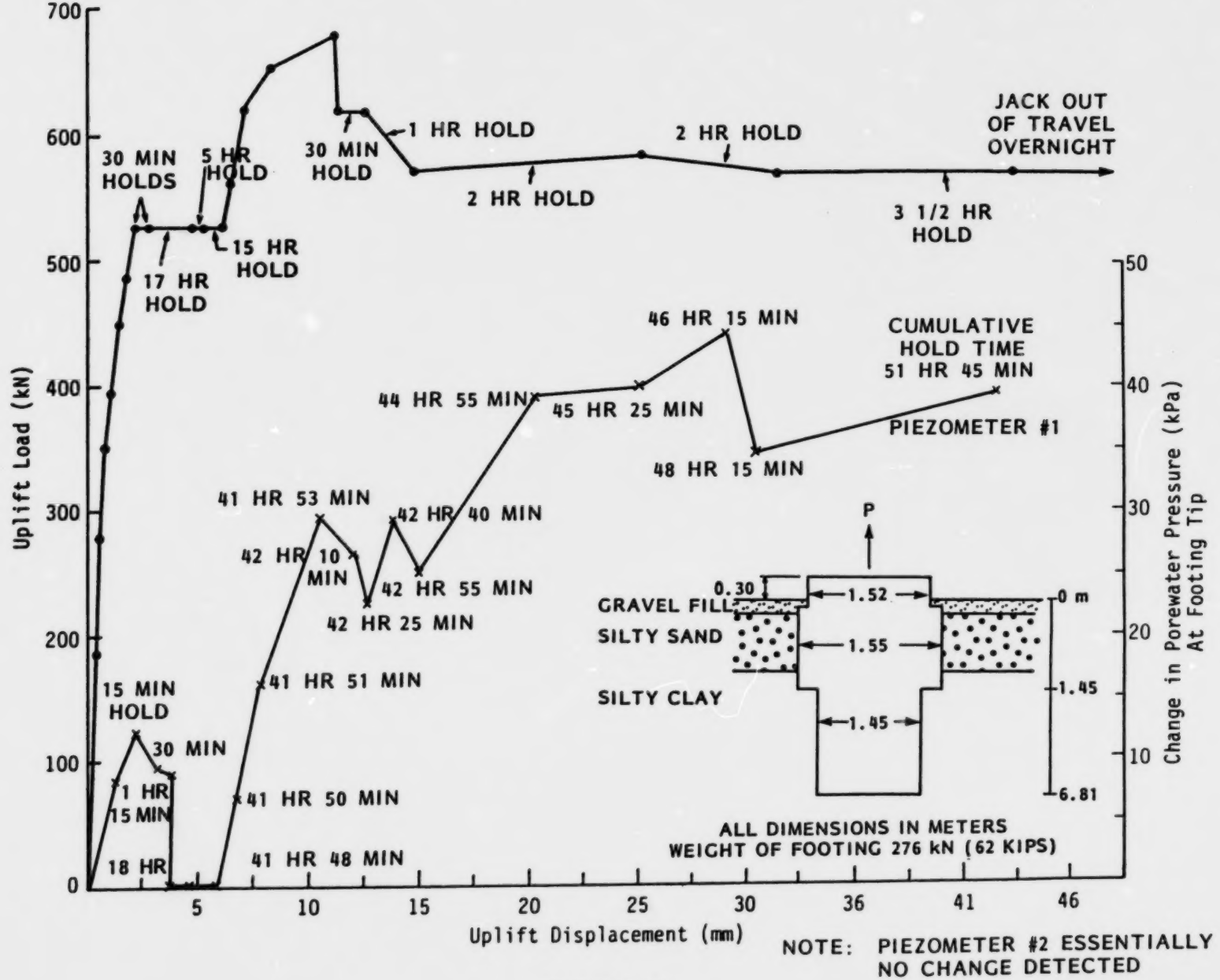


FIGURE 14

LONG TERM TEST ON FOOTING #4

86-165
171298 RD

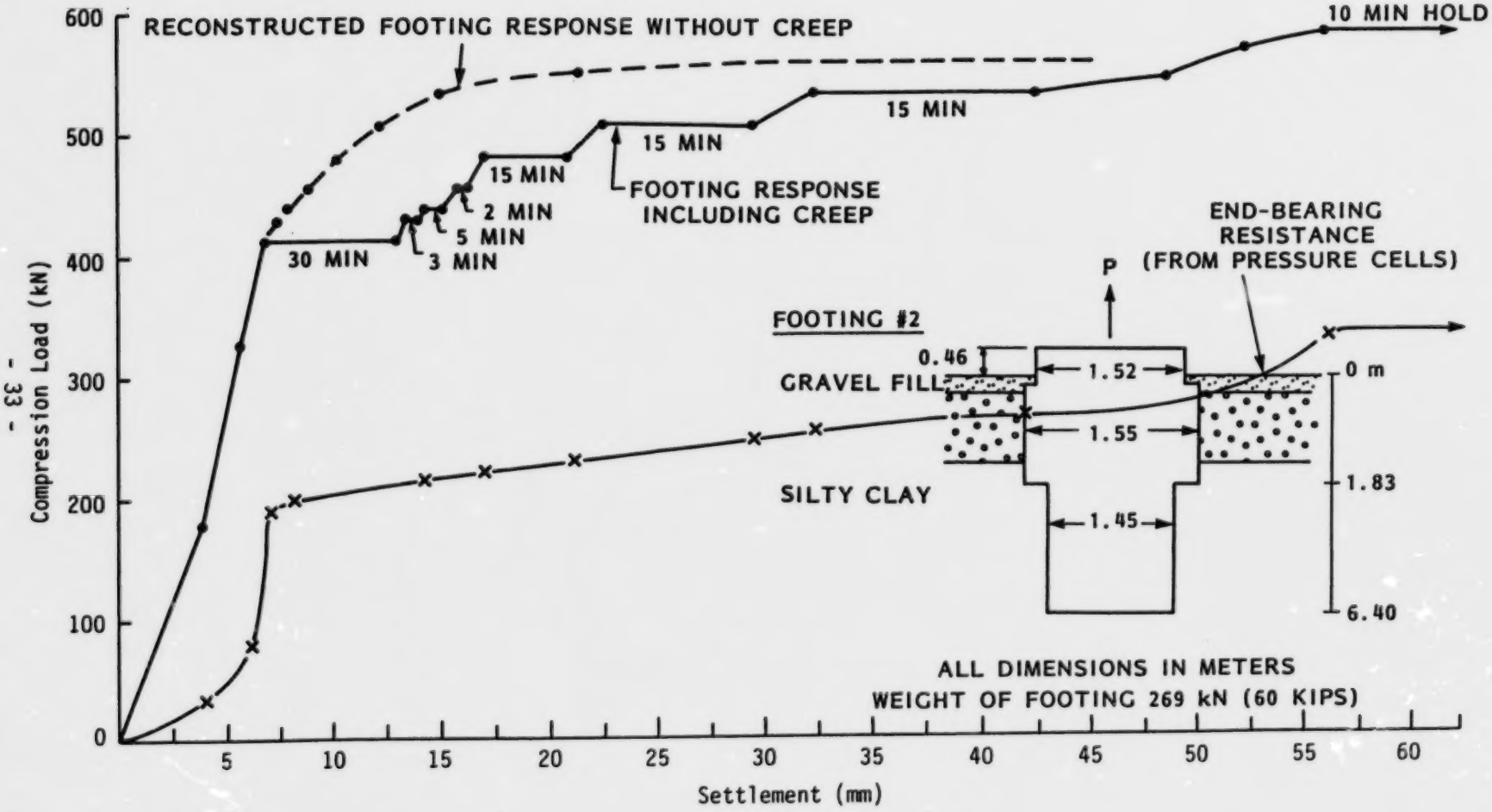


FIGURE 15

TOTAL LOAD AND END BEARING RESISTANCE VERSUS
THE SETTLEMENT OF COMPRESSION FOOTING 2

86-165
171299 RD

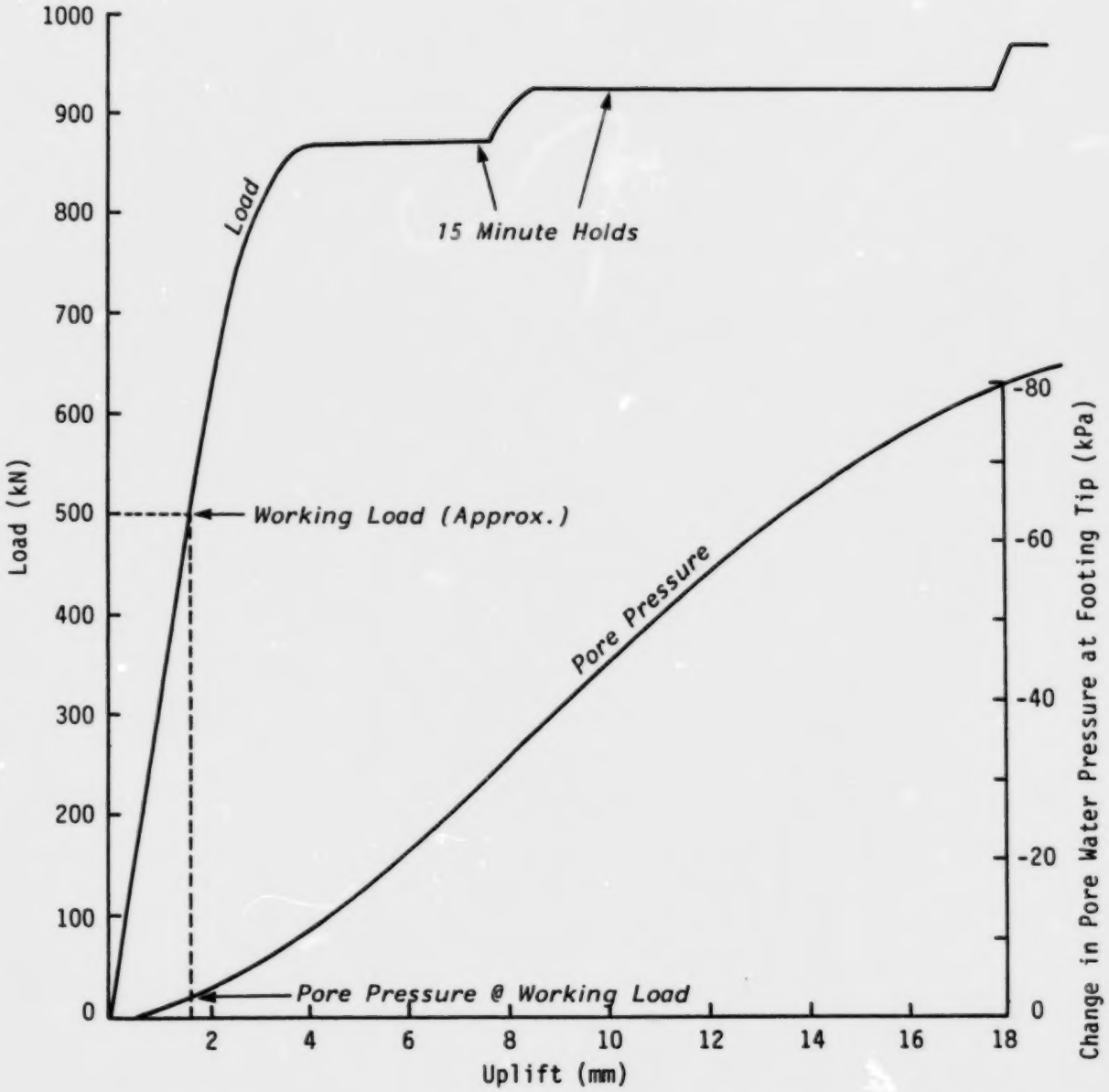


FIGURE 16
UPLIFT TEST ON FOOTING #9

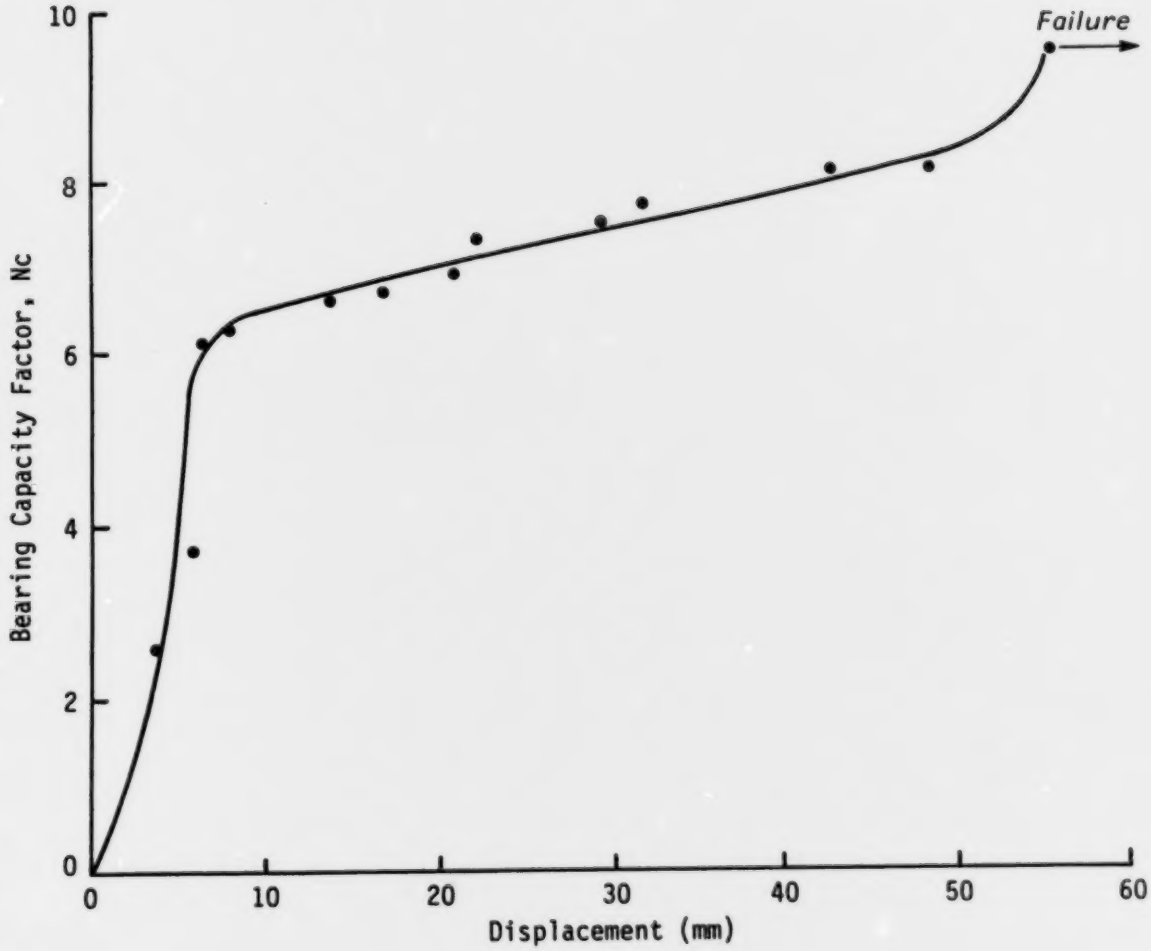


FIGURE 17
MOBILIZED BEARING CAPACITY FACTOR, Nc

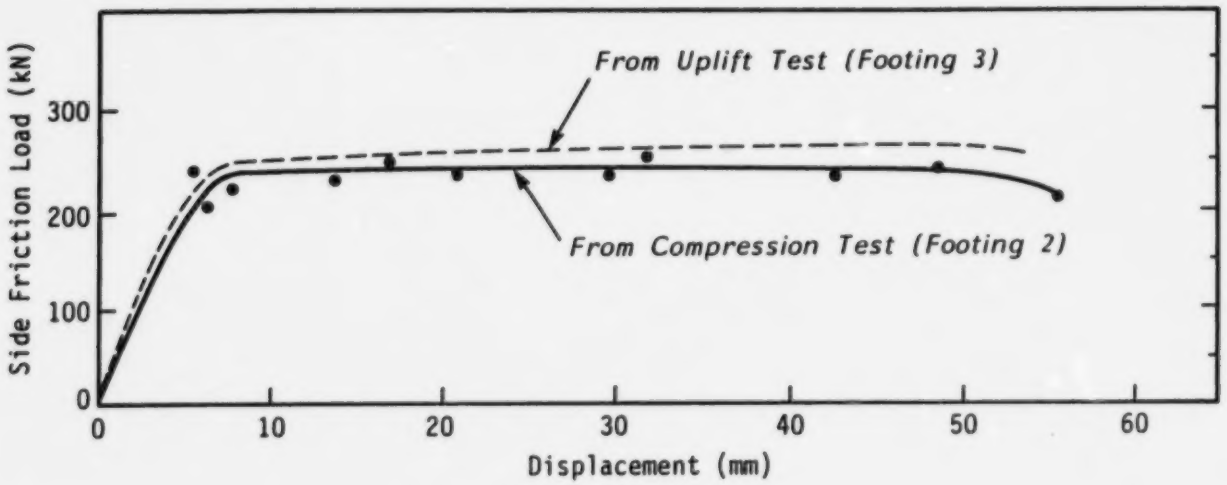


FIGURE 18

SIDE FRICTION LOAD VS FOOTING DISPLACEMENT
COMPRESSION TEST ON FOOTING #2

APPENDIX A

SOIL TEST DATA

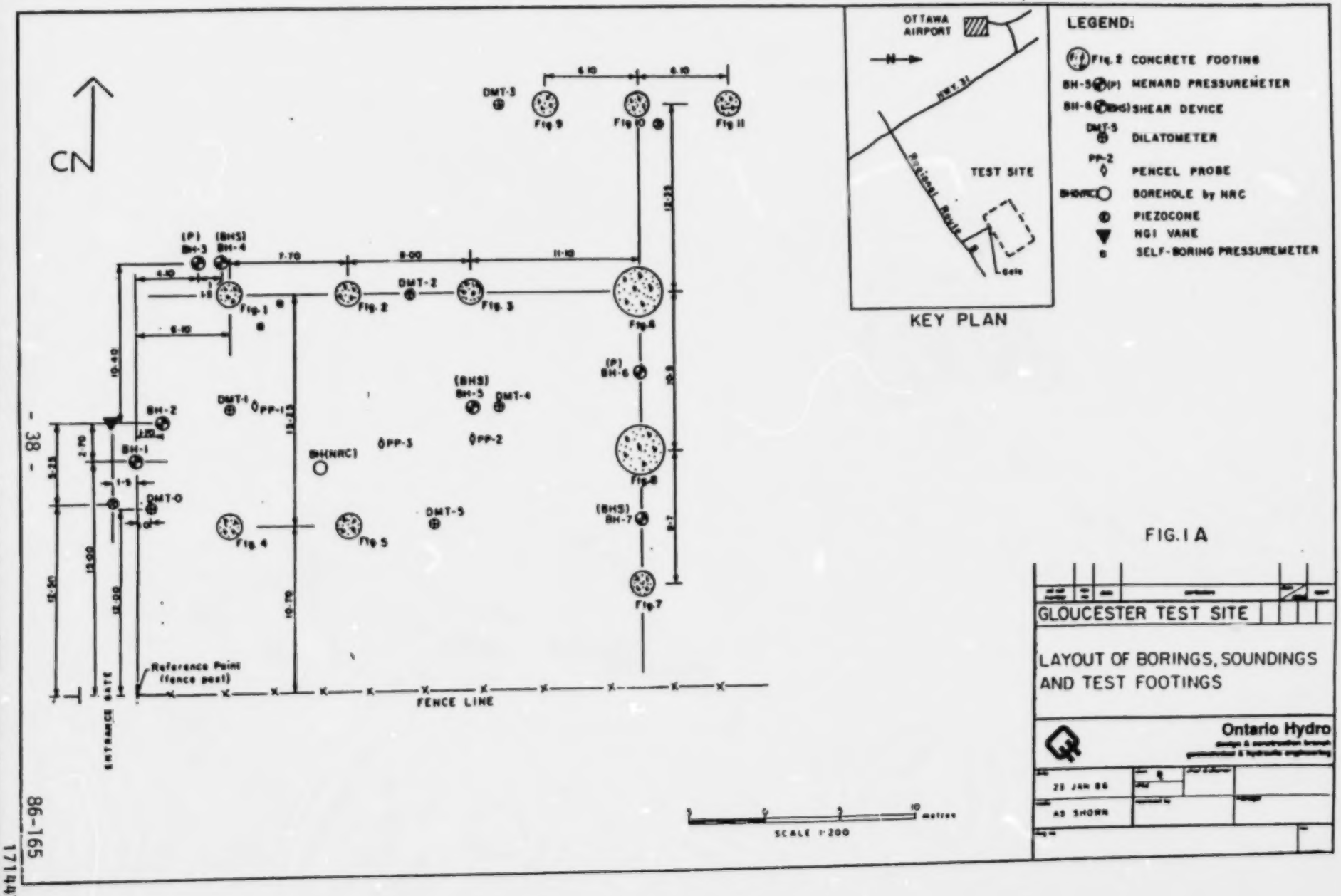


FIG. 1 A

GLOUCESTER TEST SITE

LAYOUT OF BORINGS, SOUNDINGS AND TEST FOOTINGS

Ontario Hydro

23 JAN 86

AS SHOWN

- 38 -

86-165

171447 RD

project EVALUATION OF IN-SITU TEST METHODS
location GLOUCESTER - NRC SITE
—
datum —
dip 90°
bearing —
hole type H.S. AUGER
hole size 6 1/4"
drilled 19 JUN 85
drawn 16 JAN 86 GWG
observer D. FRASER
compiled N. BECHAI
checked
approved

boring data: elev / depth	(1) sample no.	(2) sample type	(3) 'N' value per 0.3 m / ROD - %	(4) water level and casing	boring log: (5) description	(6) remarks	water inf.: (7) observation well data	(8) 'k' soil/rock cm/s: water press. test	field tests: (9) special tests: type	results	laboratory tests: natural density kg/m³	moisture content - %	(10) shear strength kPa: type	results	(11) special tests: type	results %
				W.R.	DATUM-GROUND SURFACE											
0.00					GRAVEL FILL											
0.45	S1	PUSH			COMPACT, WET, GREY-BROWN SANDY SILT, TRACE BREAKERS TO 1.0m	W.L. 2.0m AFTER COMPLETION						21.1				
0.85	S2A	12				BH. S CAVE ON COMPLETION						23.5				
1.30					SOFT, WET, BROWNISH GREY SILTY CLAY											
1.80	S3	PUSH							F.V.	21.1 (s=2.0)	1588	85.1	C/U	14.4	LL PI	44.1 25.0
3.00	S4	PUSH			FIRM, WET, GREY SILTY CLAY				F.V.	28.4 (s=2.5)	1601	59.4	C/U	16.4	LL PI	41.2 23.5
	S5	PUSH							F.V.	24.8 (s=6.8)	1505	87.1	C/U	22.1	LL PI	49.6 29.6
	S6	PUSH										81.7			LL	48.2
									F.V.	12.4 (s=1.4)		78.3 73.8			PI	28.1
	S7	PUSH									1622	70.3	C/U	30.5	LL PI	44.9 23.7
	S8	PUSH							F.V.	42.2 (s=5.7)	1440	92.9	C/U	26.0	LL PI	50.7 24.8
	S9	PUSH							F.V.	47.1 (s=6.3	1468	87.3	C/U	32.8	LL PI	54.0 29.5
	S10	PUSH							F.V.	47.1 (s=8.3)	1525	77.3	C/U	42.1	LL PI	53.2 31.3
					NOTES 1. ALL SAMPLES ARE 45.0mm ID SHELBY TUBES 2. BOREHOLE BACKFILLED											
	S11	PUSH							F.V.	52.1 (s=3.0)	1656	54.9	C/U	56.2	LL PI	38.6 18.9
	S12	PUSH							F.V.	52.1 (s=3.0)	1622	64.8	C/U	66.3	LL PI	45.8 21.3
18.80					FIRM, WET, GREY VARVED SILTY CLAY											
	S13	PUSH							F.V.	52.0 (s=4.2)	1719	45.0	C/U	50.4	LL PI	39.1 20.2
20.90					VERY STIFF, MOIST, GREY, CLAYEY, SILT TILL											
	S14	PUSH							F.V.	88.2 (s=1.4)	1976	26.9	C/U	155.0	LL PI	22.2 5.2
21.80					END OF BOREHOLE											

241956 rev. 86-06
Ontario Hydro
Design & Construction Branch
Geotechnical & Hydraulic Engineering
geotechnical data sheet
hole no.
fig 2A
sheet

project	EVALUATION OF IN-SITU TEST METHODS	hole type	H.S. AUGER	observer	D. FRASER
location	GLOUCESTER - NRC SITE	hole size	8 inch	compiled	M. BECHAI
	— dip 90	drilled	20 JUN 85	checked	
datum	— bearing —	drawn	18 JAN 86 GWG	approved	

boring data: scale - metres	elev / depth	(1) sample no.	(2) sample type	(3) "N" value per 0.3 m / ROD - %	(4) water level and coring: W.R.	boring log: (5) description	(6) remarks	water lvl.: (7) observation well data	(8) 'k' soil/rock cm/s: water press. test	field tests: (9) special tests: type	results	laboratory tests: natural density kg/m³	moisture content - %	(10) shear strength kPa: type	results	(11) special tests: type	results
						DATUM-GROUND SURFACE											
						GRAVEL FILL											
	0.40					COMPACT, WET, GREY-BROWN SANDY SILT											
	1.80					FIRM TO SOFT, GREY SILTY CLAY											
				PUSH								1674	44.5	DS	18.0		
				PUSH								1637	58.8	C/U	13.0		
				PUSH								1526	87.5	DS	20.0	Pc	60.0
				PUSH								1475	73.2	C/U	9.9		
				PUSH			W.L. ON COMPLETION										
	5.00			PUSH													
				PUSH								1633	65.5	DS	19.0		
												1631	63.0	c/u	31.8		
				PUSH									58.1	DS	20.5	Pc	77.0
				PUSH								1470	90.8	C/U	26.8		
				PUSH									87.5			Pc	80.0
				PUSH								1508	89.8	DS	30.0		
				PUSH								1485	84.8	C/U	37.9		
				PUSH													
				PUSH													
				PUSH								1552	73.8	C/U	56.8		
	12.80					END OF BOREHOLE											

NOTES
1. BOREHOLE BACKFILLED ON COMPLETION
2. ALL SOIL CONTACTS ARE APPROXIMATE
3. ALL SAMPLES ARE 60mm I.D. OBTAINED WITH GREGORY TYPE SAMPLER

LEGEND
DS SHEAR STRENGTH FROM DIRECT SHEAR TEST AT A NORMAL LOAD EQUIVALENT TO THE EFFECTIVE OVERBURDEN PRESSURE

PC PRECONSOLIDATION PRESSURE FROM STANDARD OEDOMETER TEST

Ontario Hydro
Design & Construction Branch
Geotechnical & Hydraulic Engineering

geotechnical data sheet

hole no. 3A

sheet

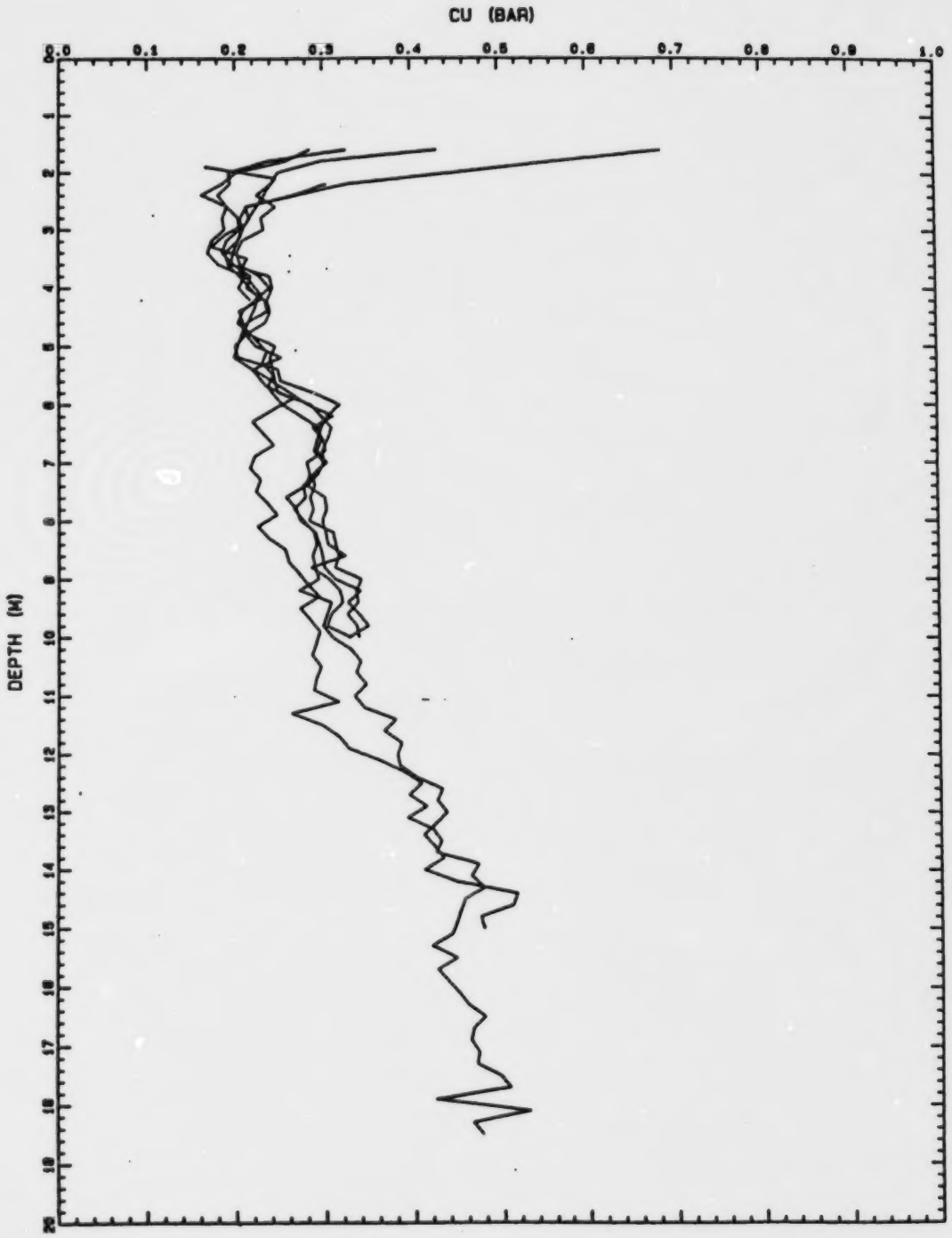


FIG. 4A DILATOMETER CU PROFILES _ GLOUCESTER

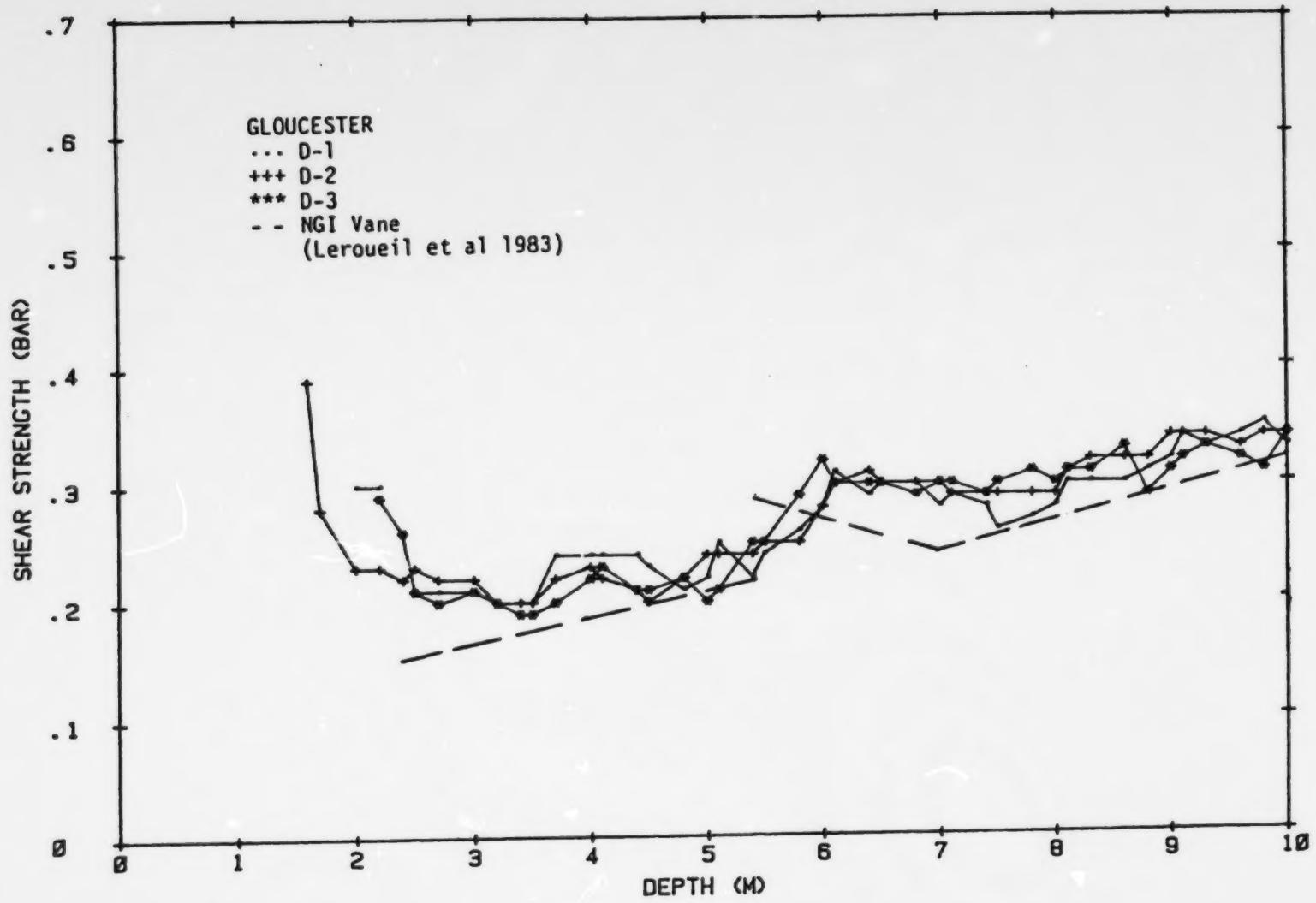


FIGURE 5A

DILATOMETER AND NGI VANE RESULTS

86-165

171455 RD

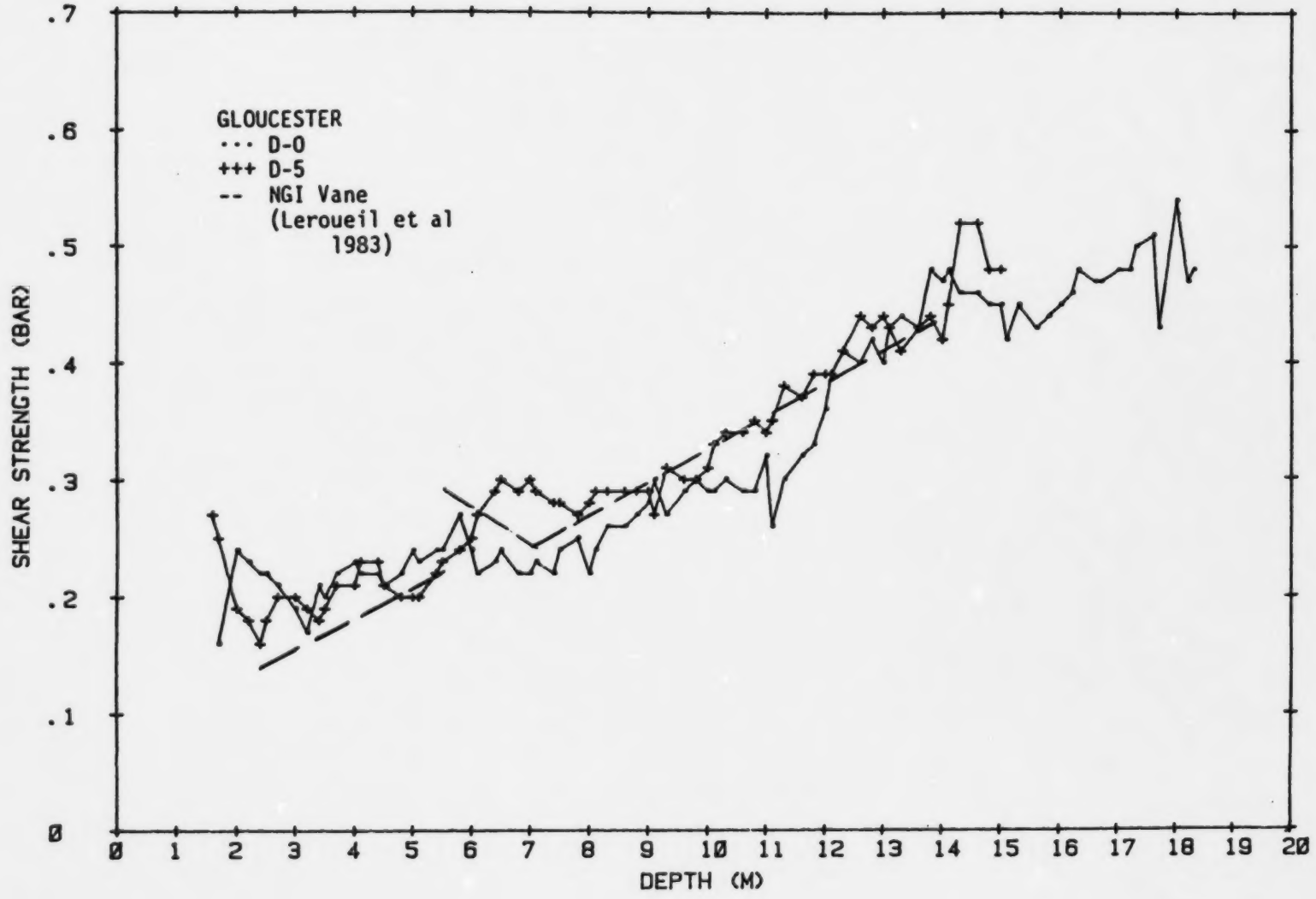


FIGURE 6A
DILATOMETER AND NGI VANE RESULTS

86-165
171457 RD

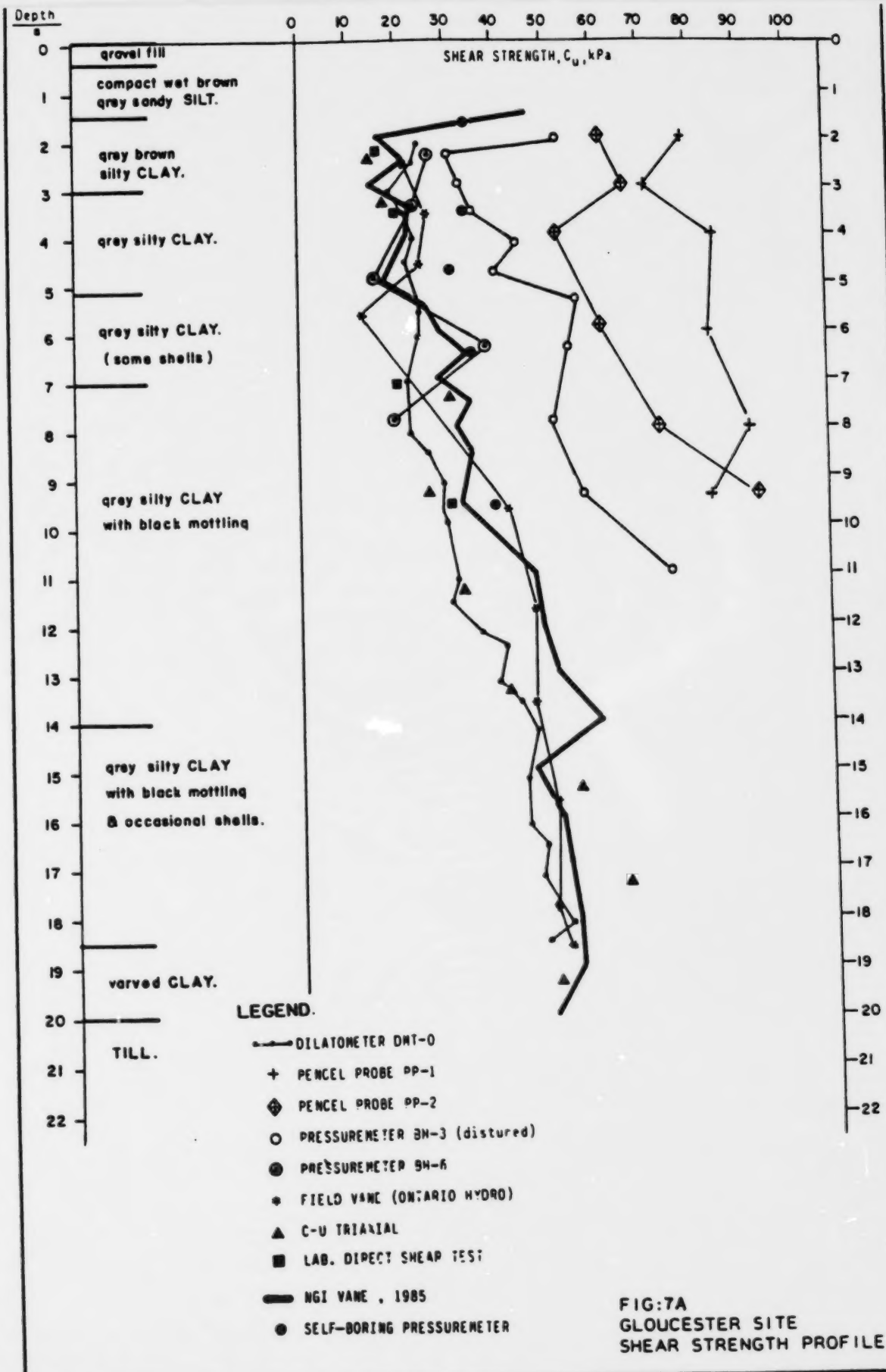


FIG:7A
GLOUCESTER SITE
SHEAR STRENGTH PROFILE

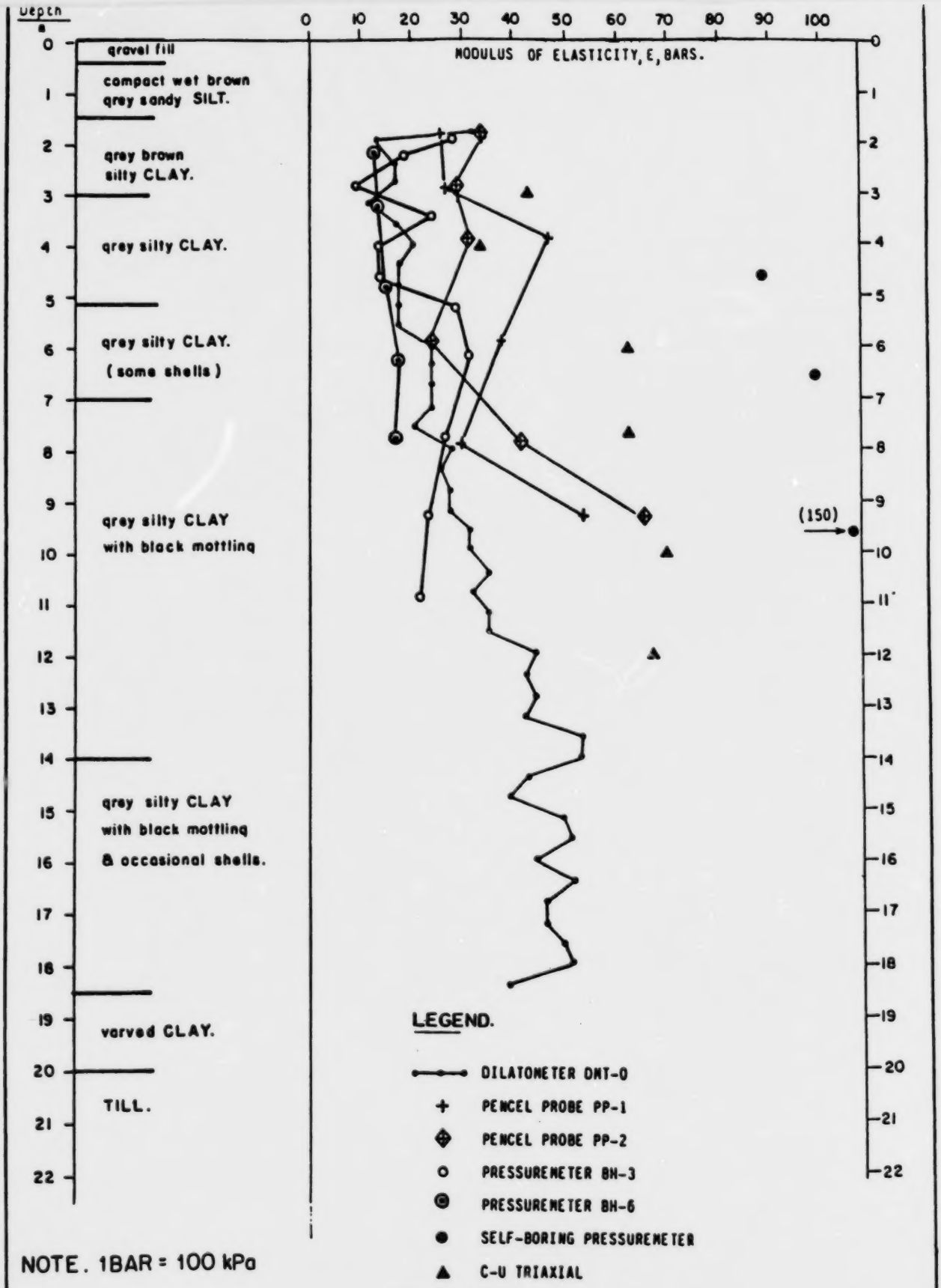


FIGURE 8A
GLOUCESTER SITE
ELASTIC MODULUS PROFILE

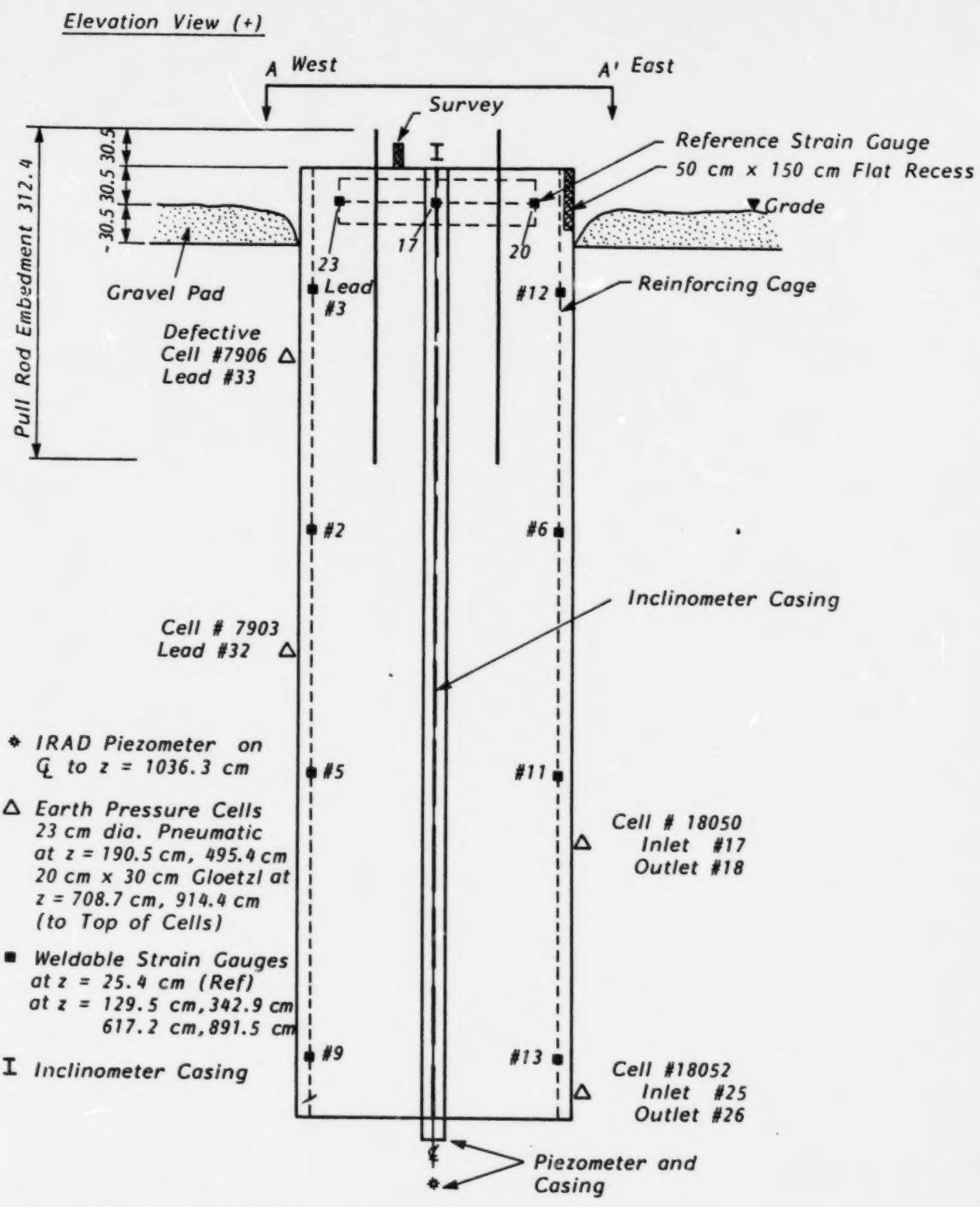
APPENDIX B

INSTRUMENTATION INSTALLATION DETAILS

TABLE B1

LIST OF FIGURE NUMBERS

Footing No	Instruments in Footing		Instruments in Soil
	Section	Plan	
1	1B	2B	3B
2	4B	5B, 6B	7B
3	8B	9B	10B
4	11B	12B	-
5	13B	14B	-
9	15B	16B	-
10	17B	18B	-
11	19B	20B	-



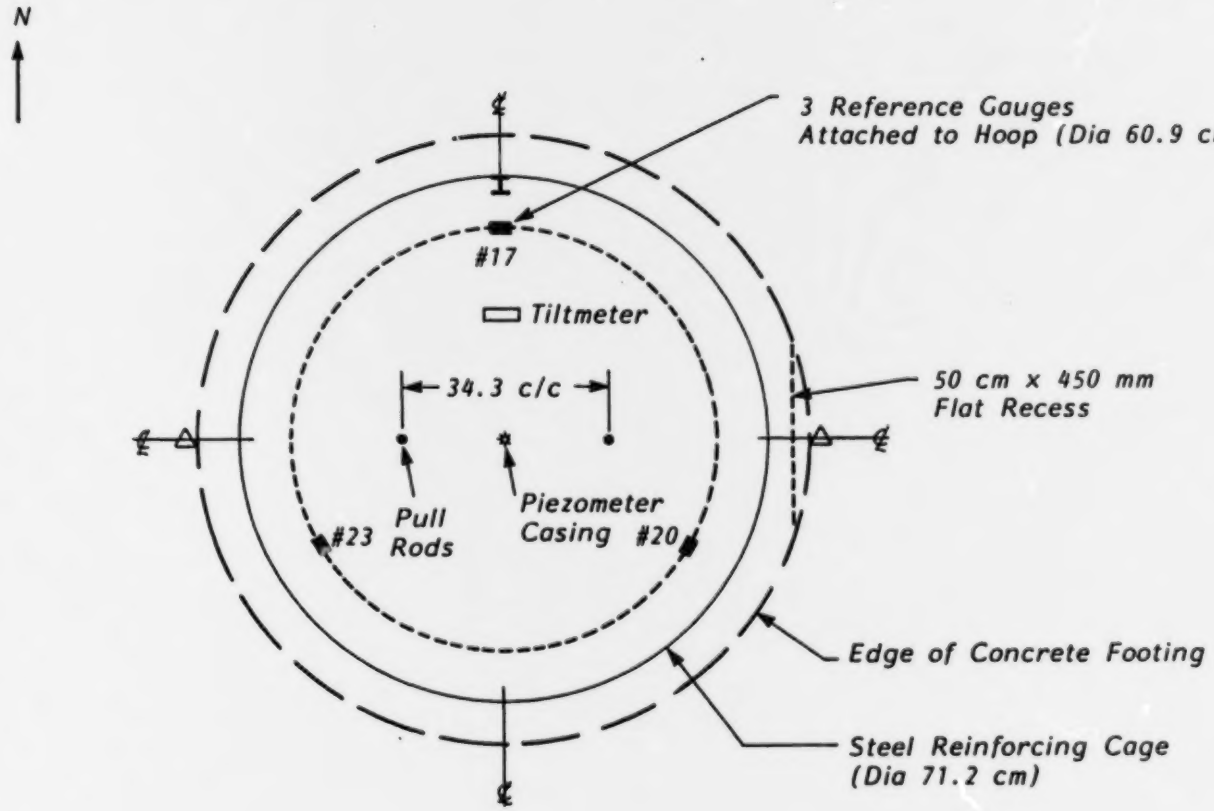
Note: (+) Not to Scale (as Dimensioned in Centimeters from Top of Footing)

FIGURE 1B

INSTRUMENTATION OF TEST FOOTING #1

0.92 m Average Diameter by 9.65 m Embedded Length

Section A-A' - Plan View (+)



- IRAD Piezometer and Casing
- △ Earth Pressure Cells
- ▬ Weldable Strain Gauges
- I Inclinometer Casing

Notes:
1) Earth Pressure Cells Oriented East-West
2) Weldable Strain Gauges Oriented East-West (except at z = 891.5 cm) on East Side was Offset 15.2 cm (6") of E-W Line)
3) West Pull Rod Embedded only 223.5 cm (88") in Concrete Since a Jacking Rig in the Cage Obstructed Further Embedment

(+) Not to Scale as Dimensioned

FIGURE 2B
INSTRUMENTATION OF TEST FOOTING #1

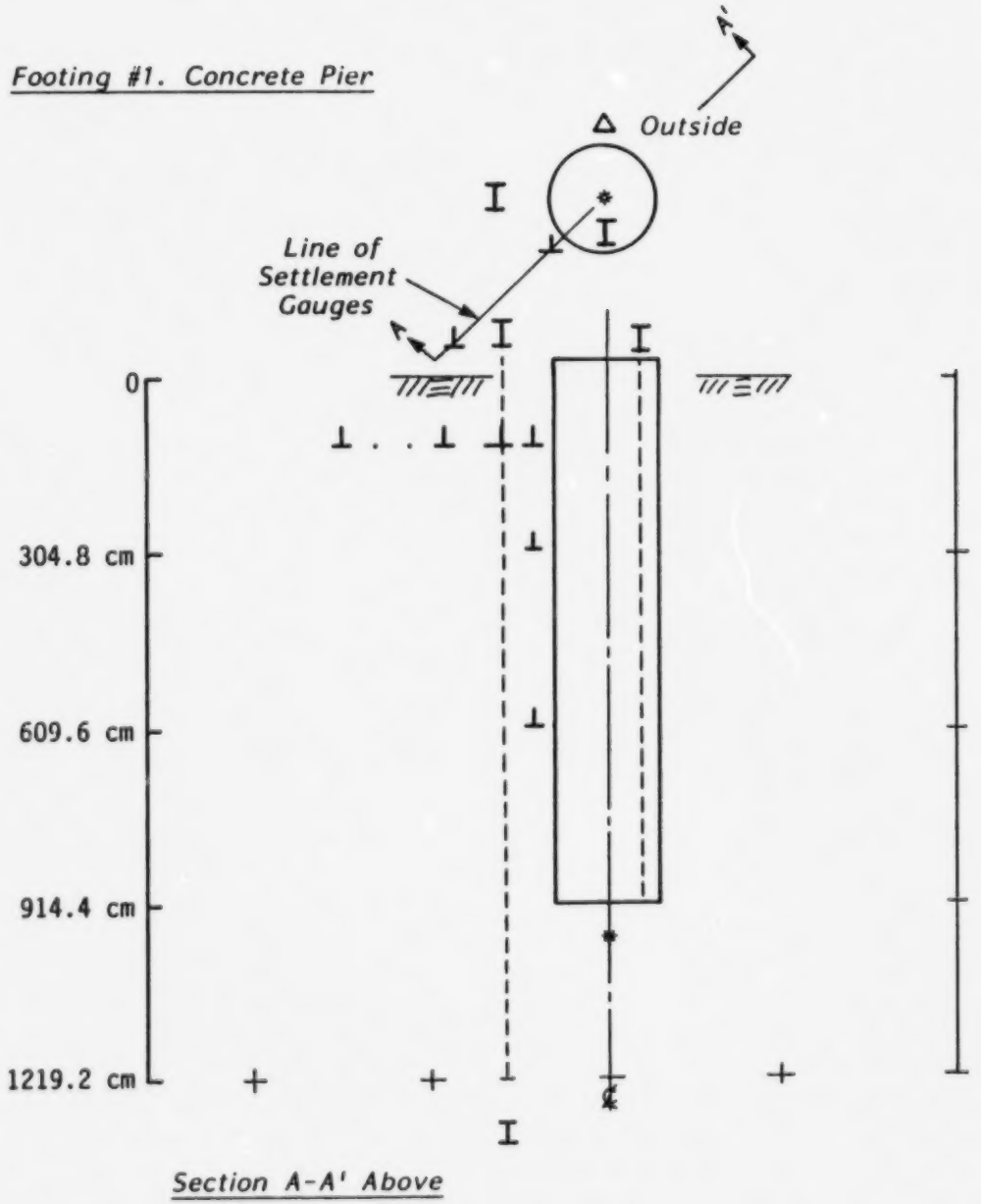
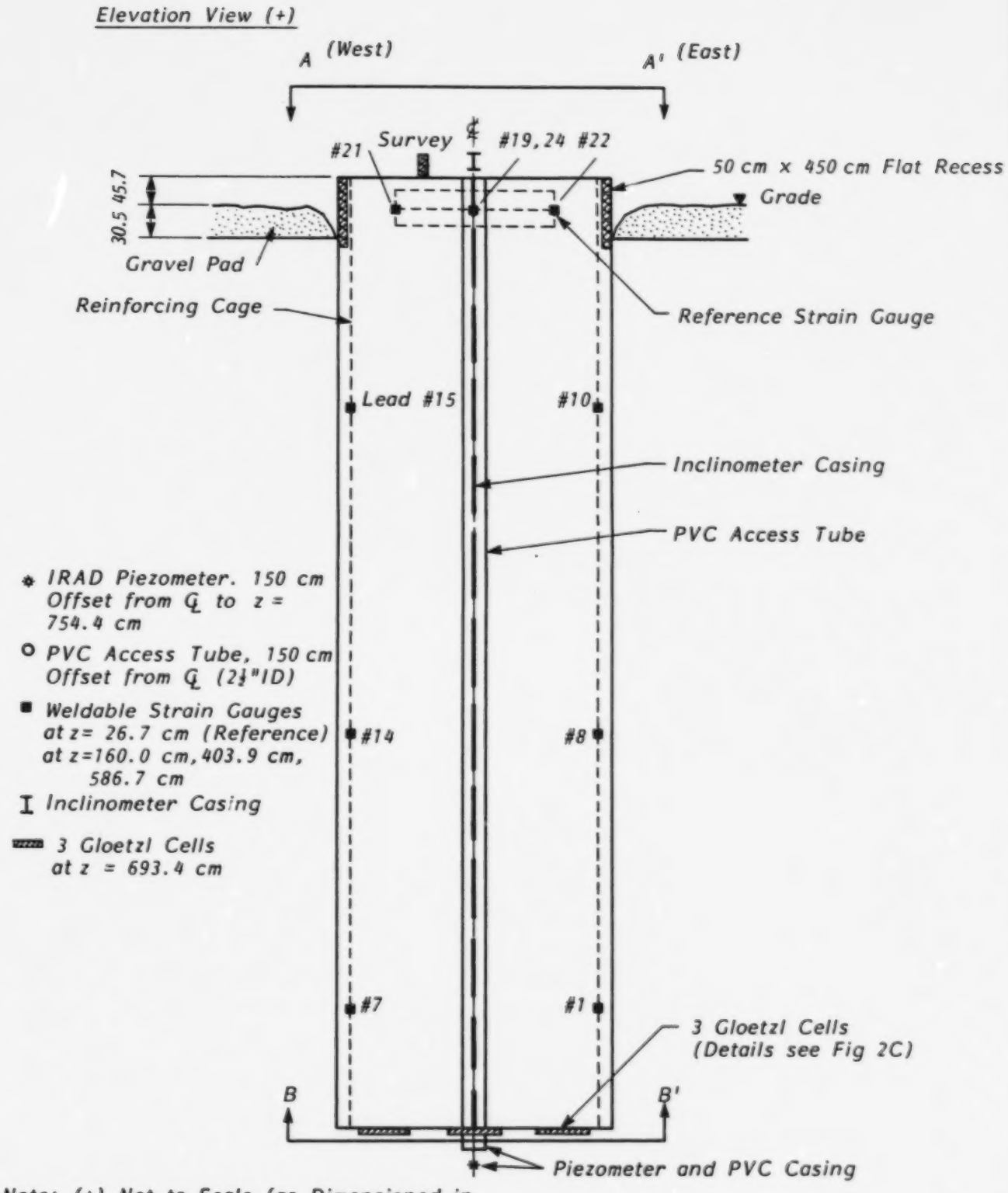

Footing #1. Concrete Pier
Outside
Line of Settlement Gauges
0
304.8 cm
609.6 cm
914.4 cm
1219.2 cm
Section A-A' Above
I Inclinometer at 46cm from Pier
Inclinometer Inside Pile
⊥ Settlement Gauges (6)
at z = 121.9cm - Dist 15.2 cm
45.7 cm
91.4 cm
182.9 cm
at z = 304.8cm - Dist 15.2 cm
at z = 609.6cm - Dist 15.2 cm
* IRAD Piezometer at z = 975.4cm at ℄
△ Earth Pressure Cell (1)
at z = 914.4 cm to Top of Cell

FIGURE 3B
INSTRUMENTATION OF SOIL BESIDE FOOTING #1

86-165
171331 RD



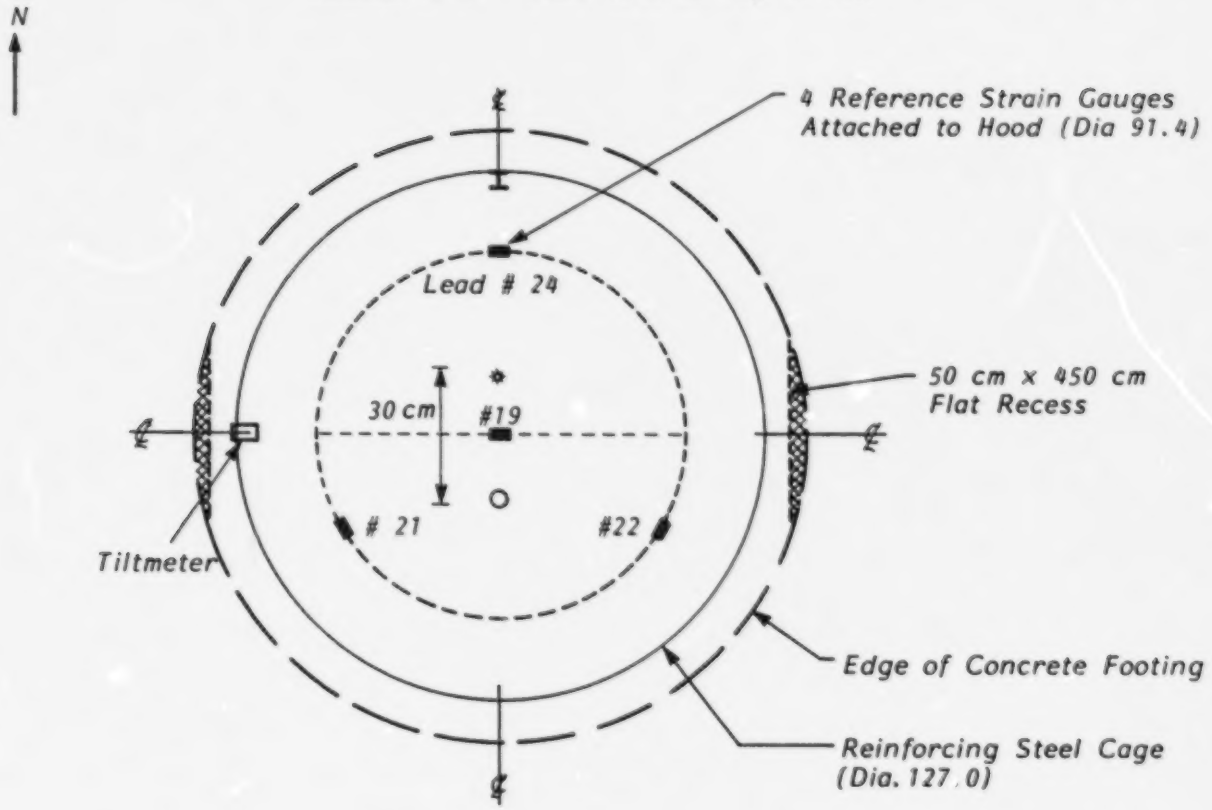
Note: (+) Not to Scale (as Dimensioned in Centimeters from Top of Footing)

FIGURE 4B

INSTRUMENTATION OF TEST FOOTING #2

1.50 m Average Diameter x 6.40 m Embedded Length

Section A-A' - Plan View of Top of Pier (+)



* IRAD Piezometer and Casing

▬ Weldable Strain Gauges

I Inclinometer Casting

O PVC Access Tube (6 cm ID)

Notes: 1) Weldable Strain Gauges Oriented East-West

(+) Not to Scale as Dimensioned

FIGURE 5B

INSTRUMENTATION OF TEST FOOTING #2

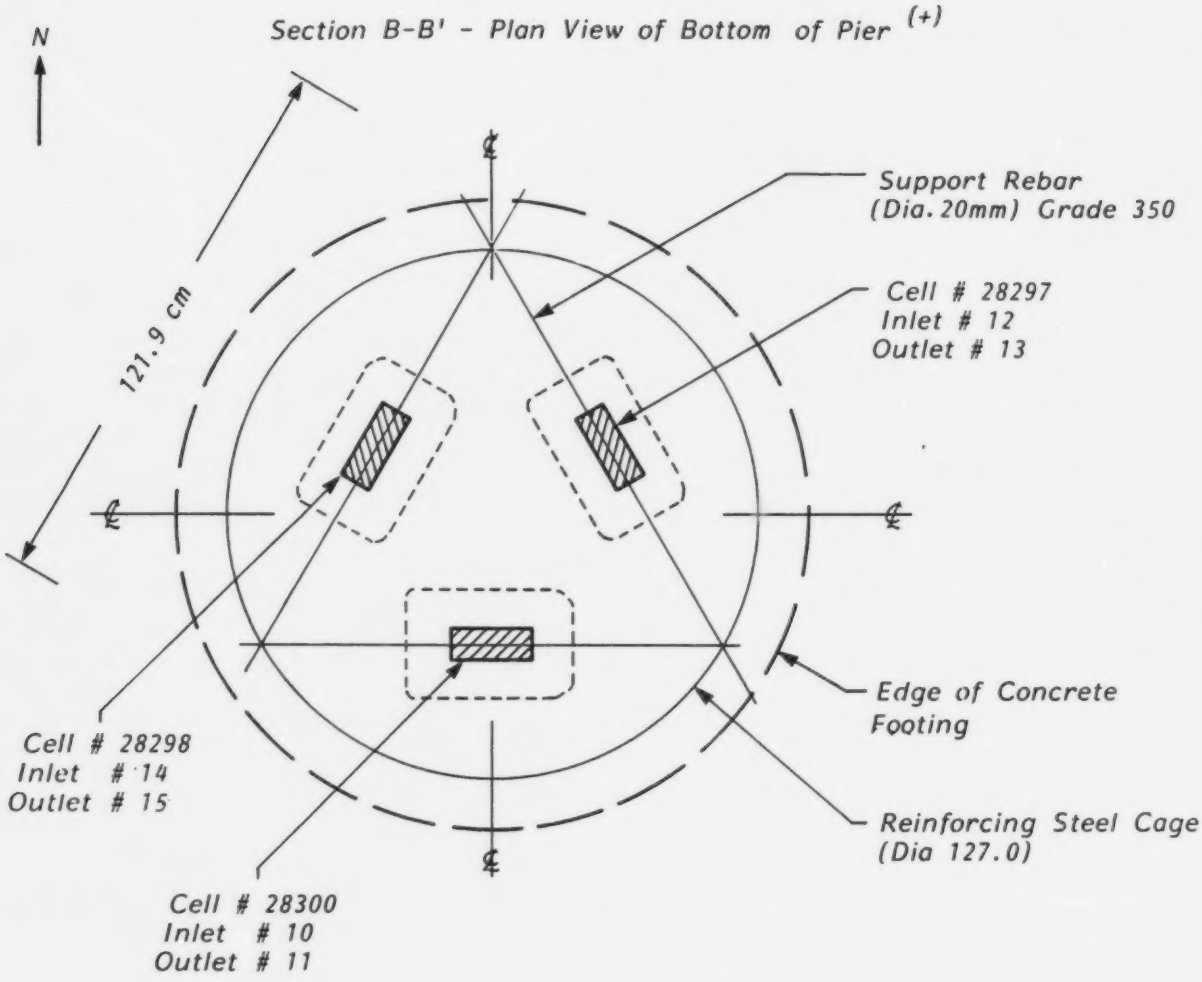


FIGURE 6B
INSTRUMENTATION OF TEST FOOTING #2

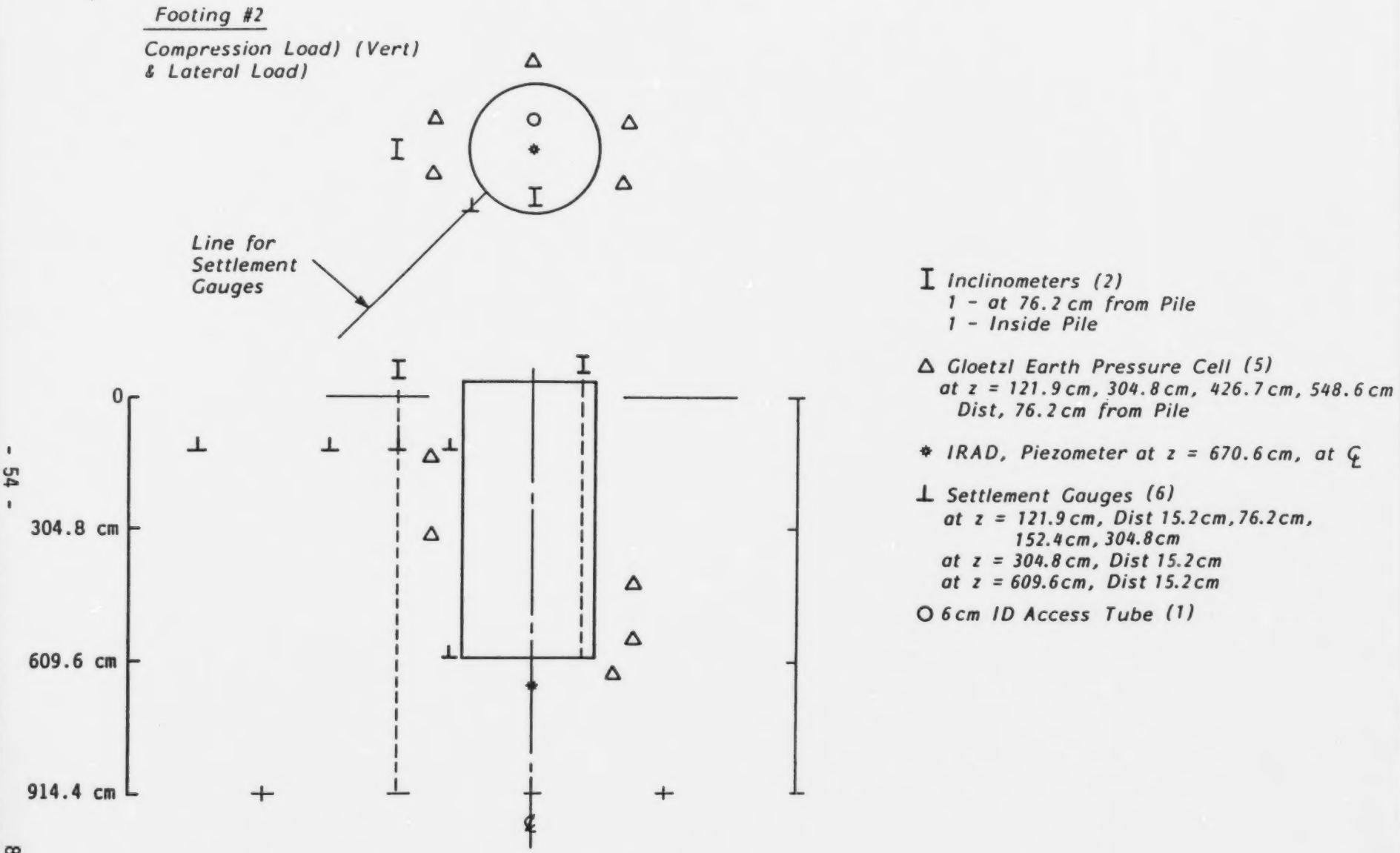

Footing #2
Compression Load) (Vert)
& Lateral Load)
Line for
Settlement
Gauges
0
304.8 cm
609.6 cm
914.4 cm
I Inclinometers (2)
1 - at 76.2 cm from Pile
1 - Inside Pile
Δ Gloetzl Earth Pressure Cell (5)
at z = 121.9 cm, 304.8 cm, 426.7 cm, 548.6 cm
Dist, 76.2 cm from Pile
* IRAD, Piezometer at z = 670.6 cm, at ℄
⊥ Settlement Gauges (6)
at z = 121.9 cm, Dist 15.2cm, 76.2cm,
152.4cm, 304.8cm
at z = 304.8 cm, Dist 15.2cm
at z = 609.6cm, Dist 15.2cm
O 6 cm ID Access Tube (1)

FIGURE 7B
INSTRUMENTATION OF SOIL BESIDE FOOTING #2

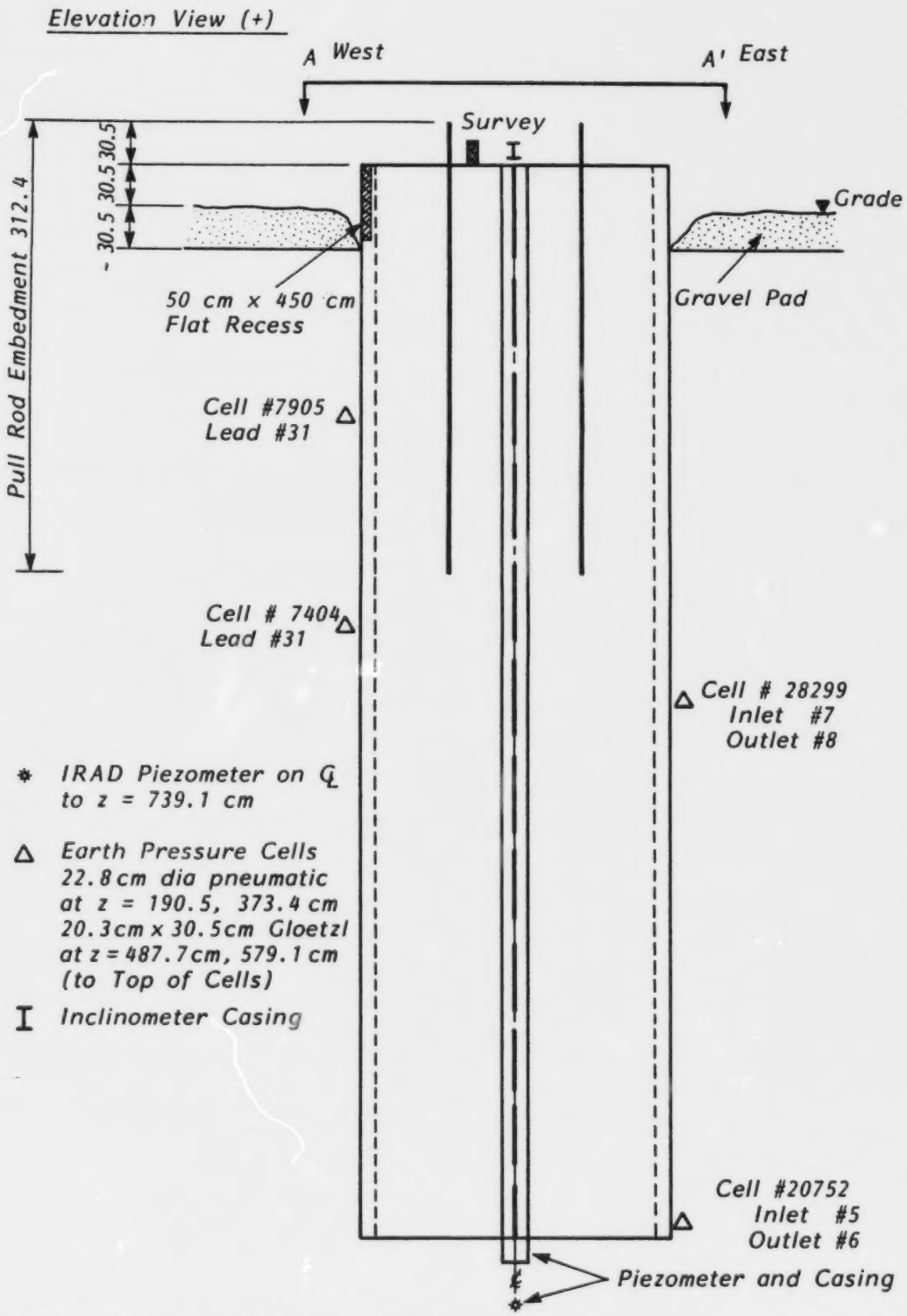
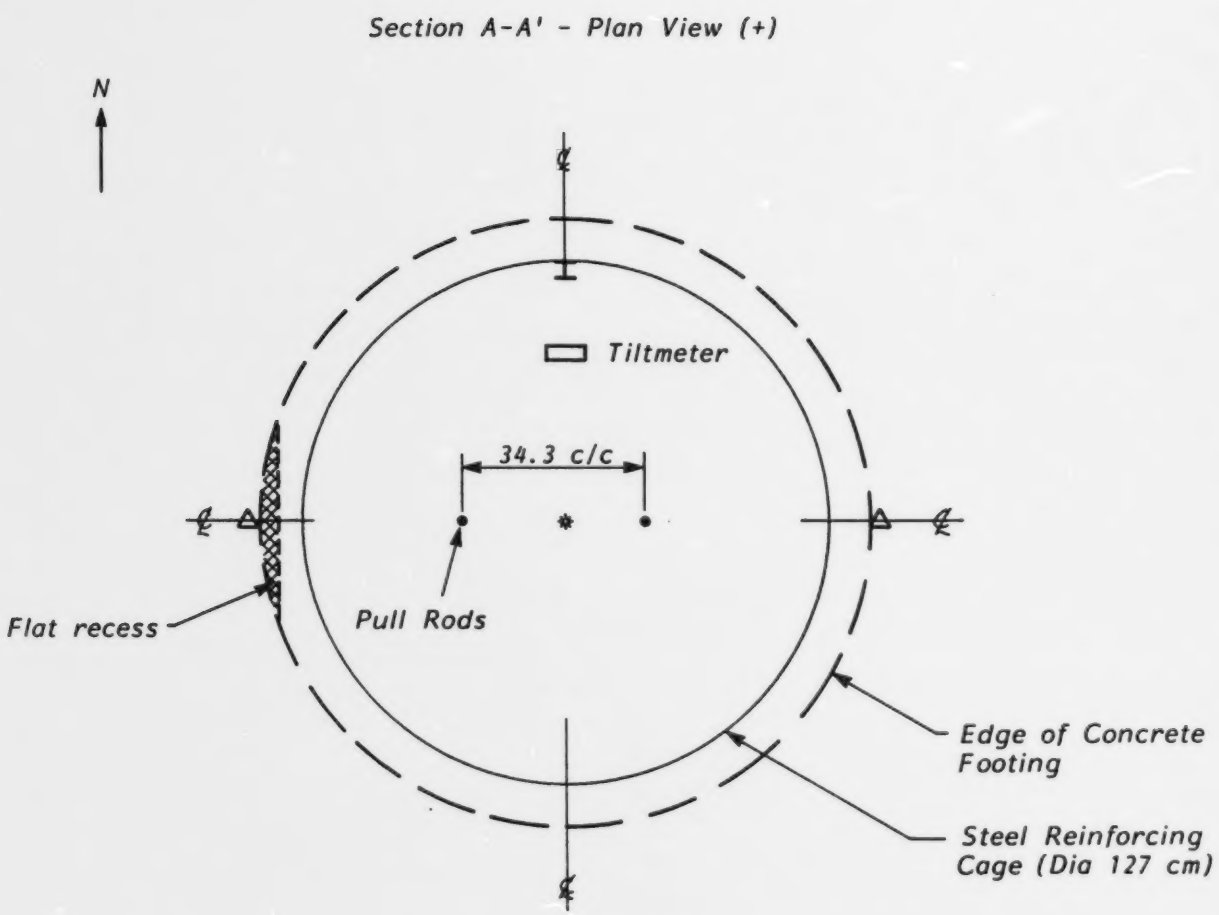


FIGURE 8B

INSTRUMENTATION OF TEST FOOTING #3

1.50 m Average Diameter x 6:40 m Embedded Length



* IRAD Piezometer and Casing

I inclinometer Casing

△ Earth Pressure Cells

Note: (+) Not to Scale as Dimensioned

FIGURE 9B
INSTRUMENTATION OF TEST FOOTING #3

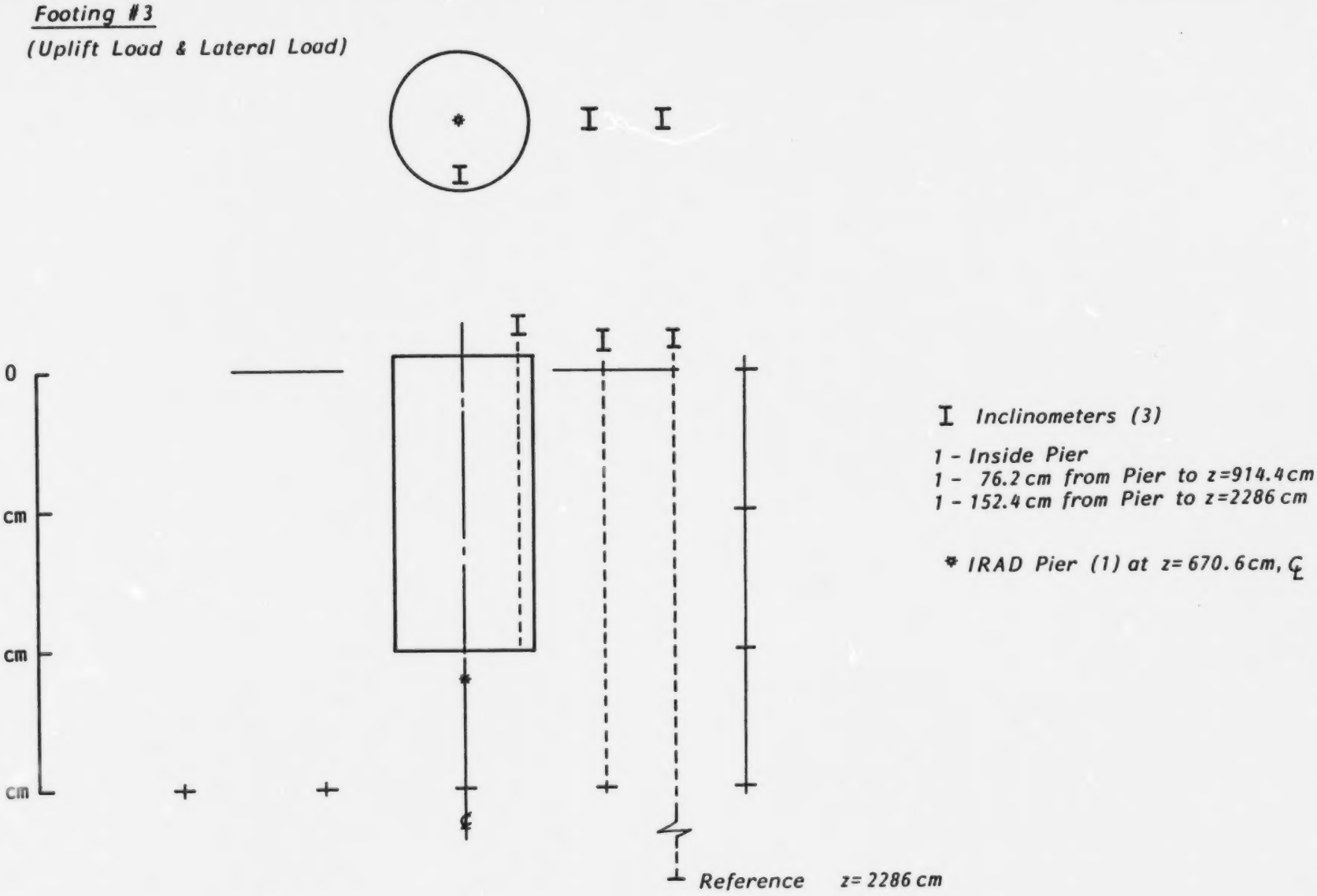

Footing #3
(Uplift Load & Lateral Load)
0
304.8 cm
609.6 cm
914.4 cm
Reference z= 2286 cm
I Inclinometers (3)
1 - Inside Pier
1 - 76.2 cm from Pier to z=914.4 cm
1 - 152.4 cm from Pier to z=2286 cm
* IRAD Pier (1) at z= 670.6 cm, ℄

FIGURE 10B
INSTRUMENTATION OF SOIL BESIDE FOOTING #3

86-165
171314 RD

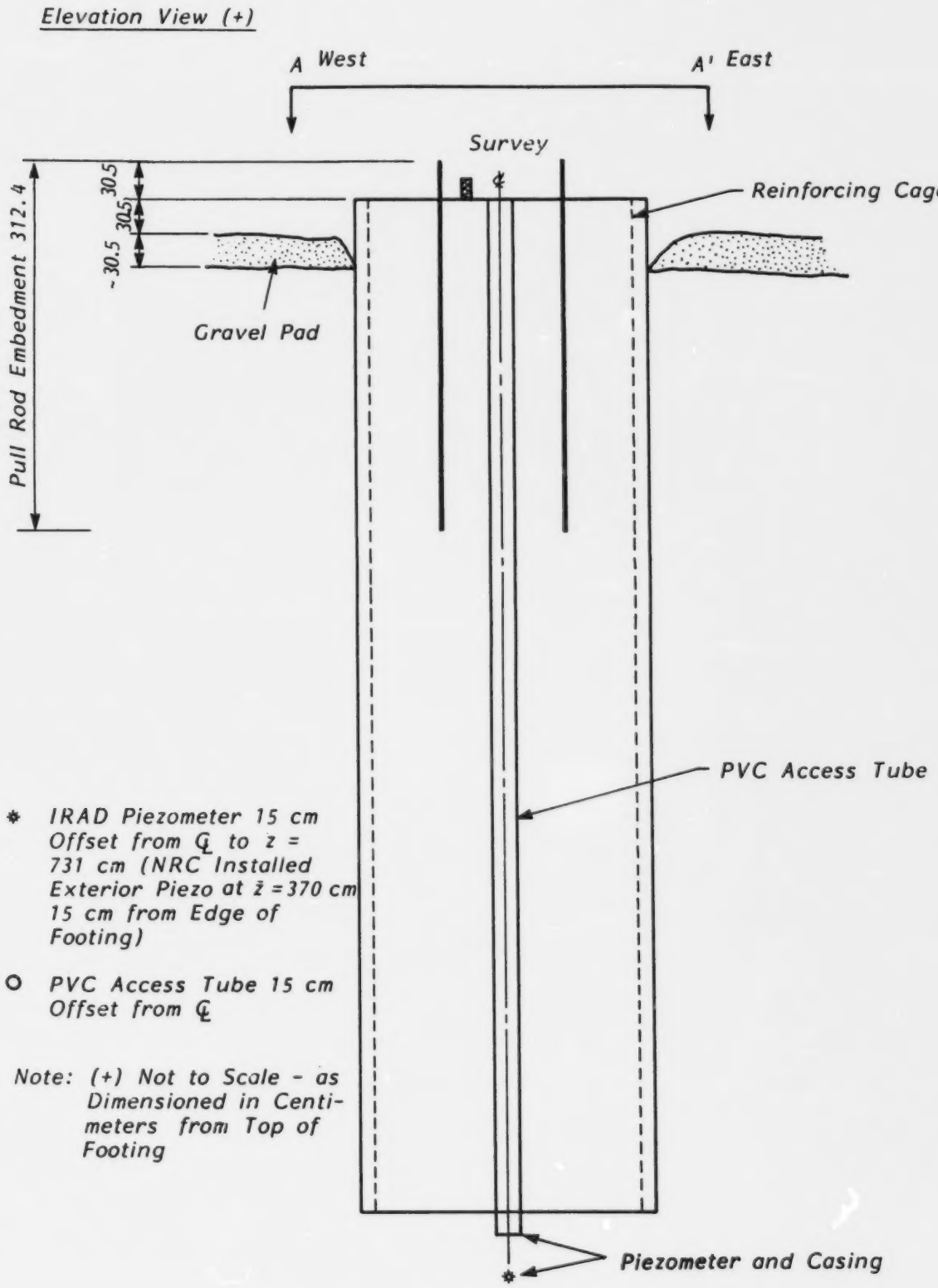
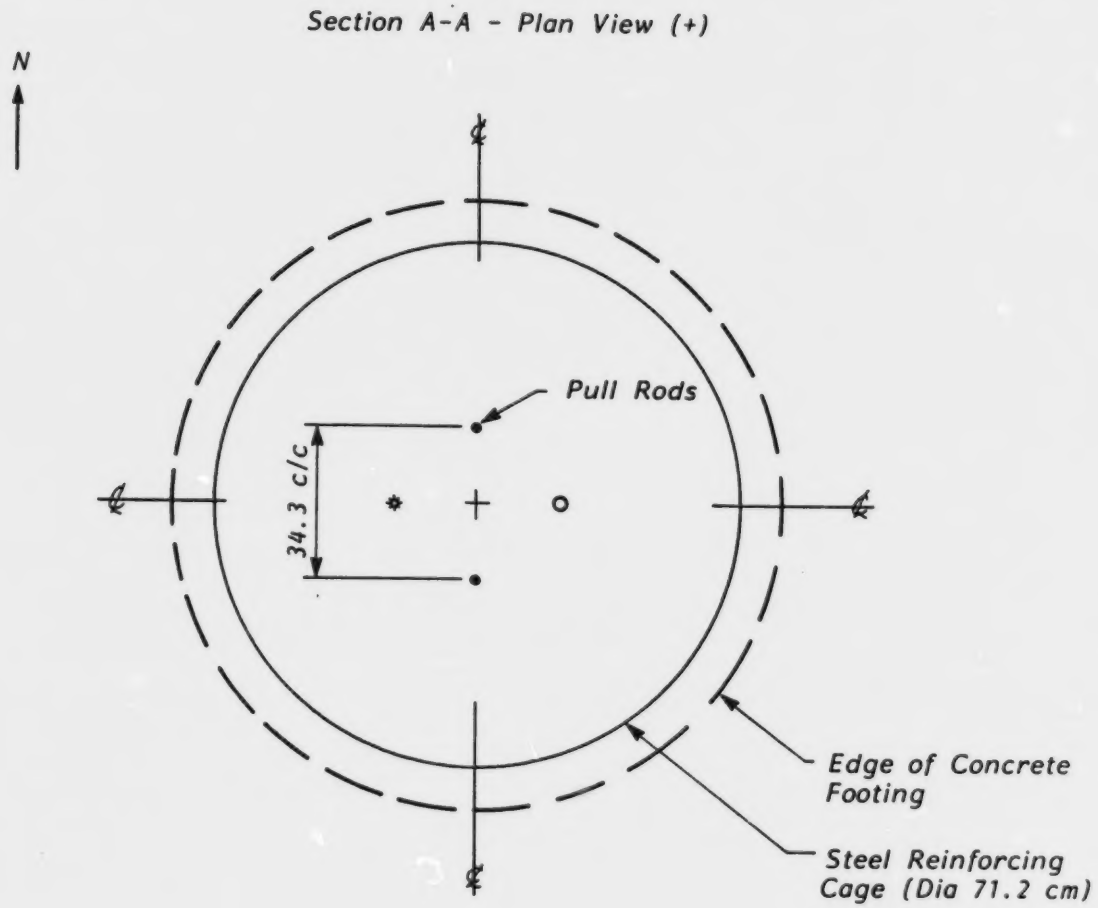

Elevation View (+)
A West
A' East
Survey
Reinforcing Cage
30.5
30.5
- 30.5
Pull Rod Embedment 312.4
Gravel Pad
PVC Access Tube
* IRAD Piezometer 15 cm Offset from ℄ to z = 731 cm (NRC Installed Exterior Piezo at z = 370 cm 15 cm from Edge of Footing)
O PVC Access Tube 15 cm Offset from ℄
Note: (+) Not to Scale - as Dimensioned in Centimeters from Top of Footing
Piezometer and Casing

FIGURE 11B

INSTRUMENTATION OF TEST FOOTING #4

150 m Average Diameter x 6.81 m Embedded Length



* IRAD Piezometer

o PVC Access Tube

Note: (+) Not to Scale - as Dimensioned

FIGURE 12B
INSTRUMENTATION OF TEST FOOTING #4

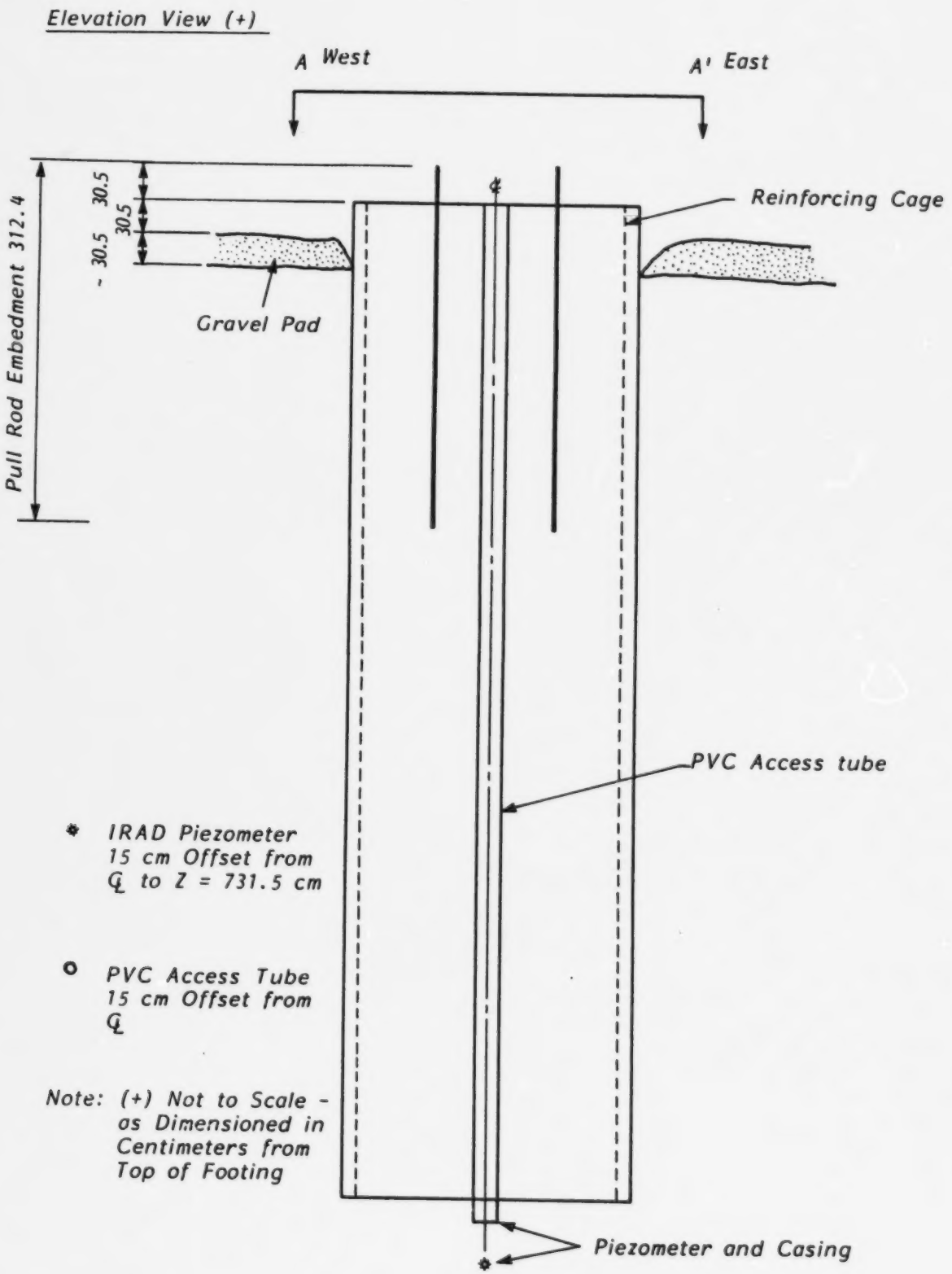


FIGURE 13B

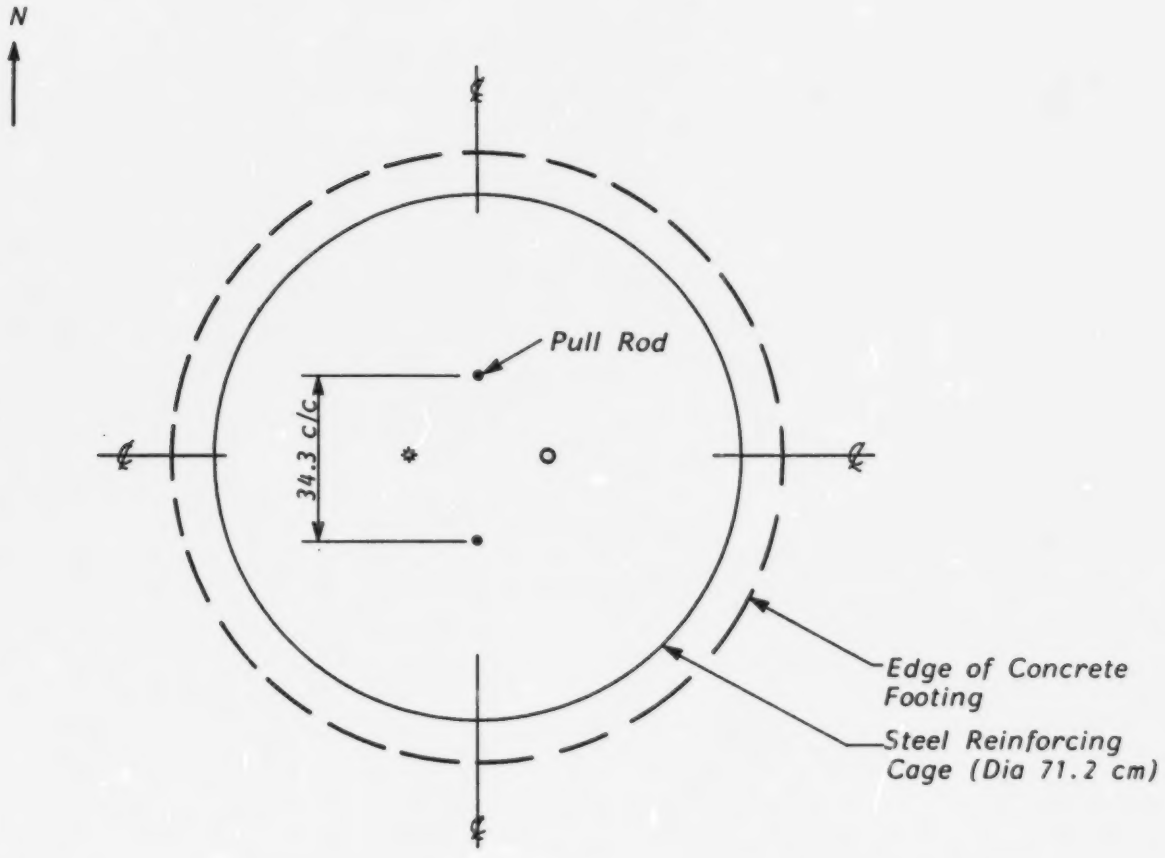
INSTRUMENTATION OF TEST FOOTING #5

150 m Average Diameter x 6.32 m Embedded Length

86-165

171317 RD

Section A-A' - Plan View (+)

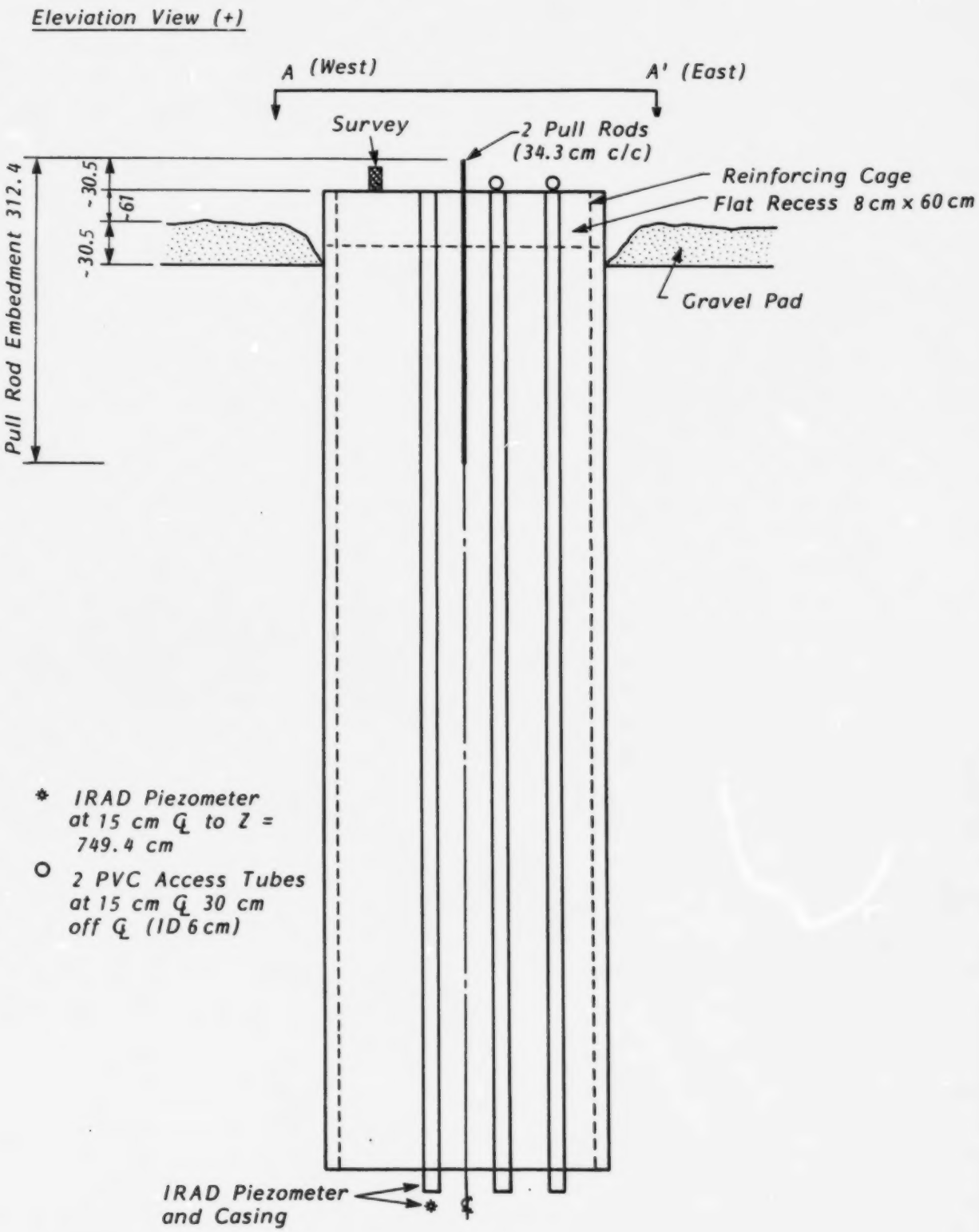


* IRAD Piezometer

o PVC Access Tube

Note: (+) Not to Scale - as Dimensioned

FIGURE 14B
INSTRUMENTATION OF TEST FOOTING #5



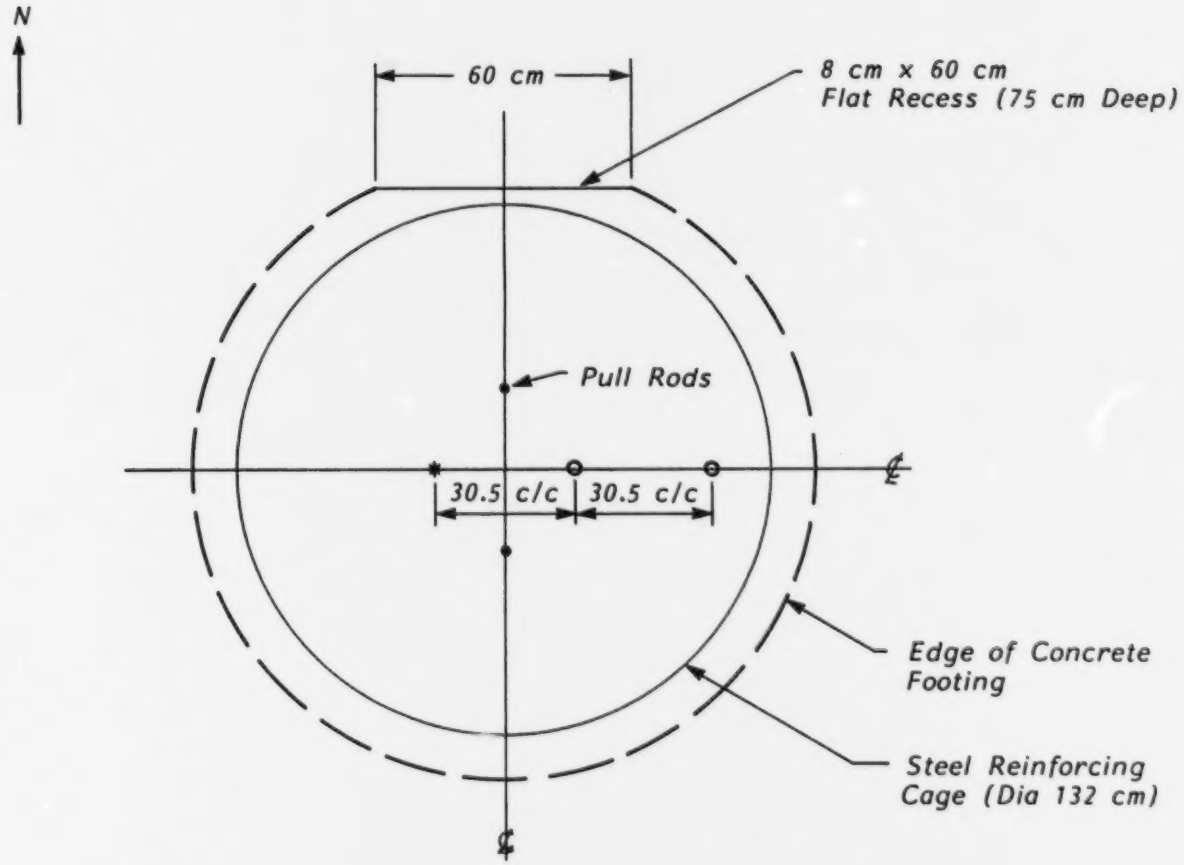
Note: (+) Not to Scale (as Dimensioned in Centimeters from Top of Footing)

FIGURE 15B

INSTRUMENTATION OF TEST FOOTING #9

1.60 m Average Diameter x 6.88 m Embedded Length

Section A-A' - Plan View (*)

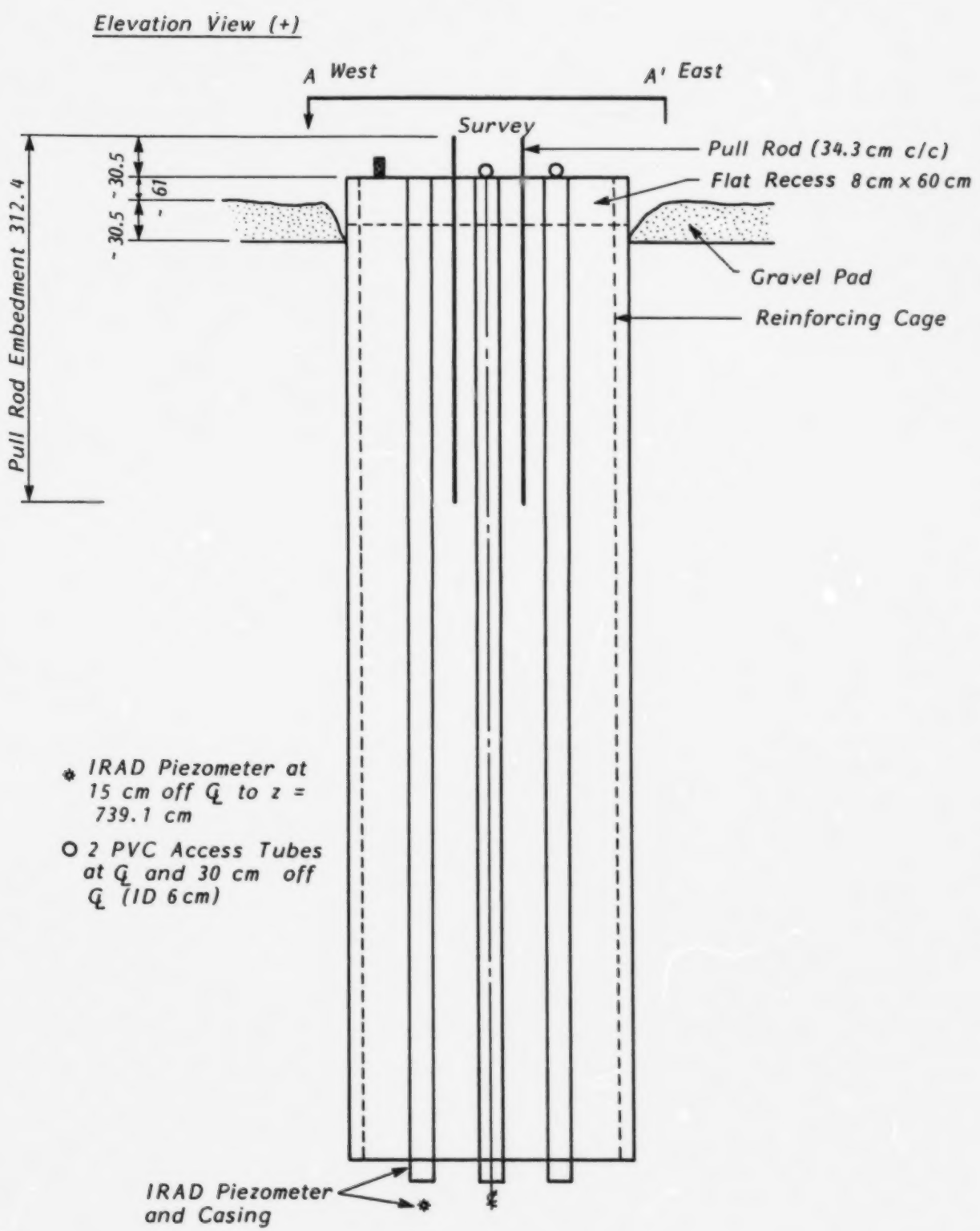


* IRAD Piezometer and Casing

o 2 PVC Access Tubes

Note: (+) Not to Scale as Dimensioned

FIGURE 16B
INSTRUMENTATION OF THE TEST FOOTING #9


Elevation View (+)
A West
A' East
Survey
Pull Rod (34.3 cm c/c)
Flat Recess 8 cm x 60 cm
Gravel Pad
Reinforcing Cage
Pull Rod Embedment 312.4
- 30.5
- 61
- 30.5
* IRAD Piezometer at 15 cm off ℄ to z = 739.1 cm
O 2 PVC Access Tubes at ℄ and 30 cm off ℄ (ID 6 cm)
IRAD Piezometer and Casing

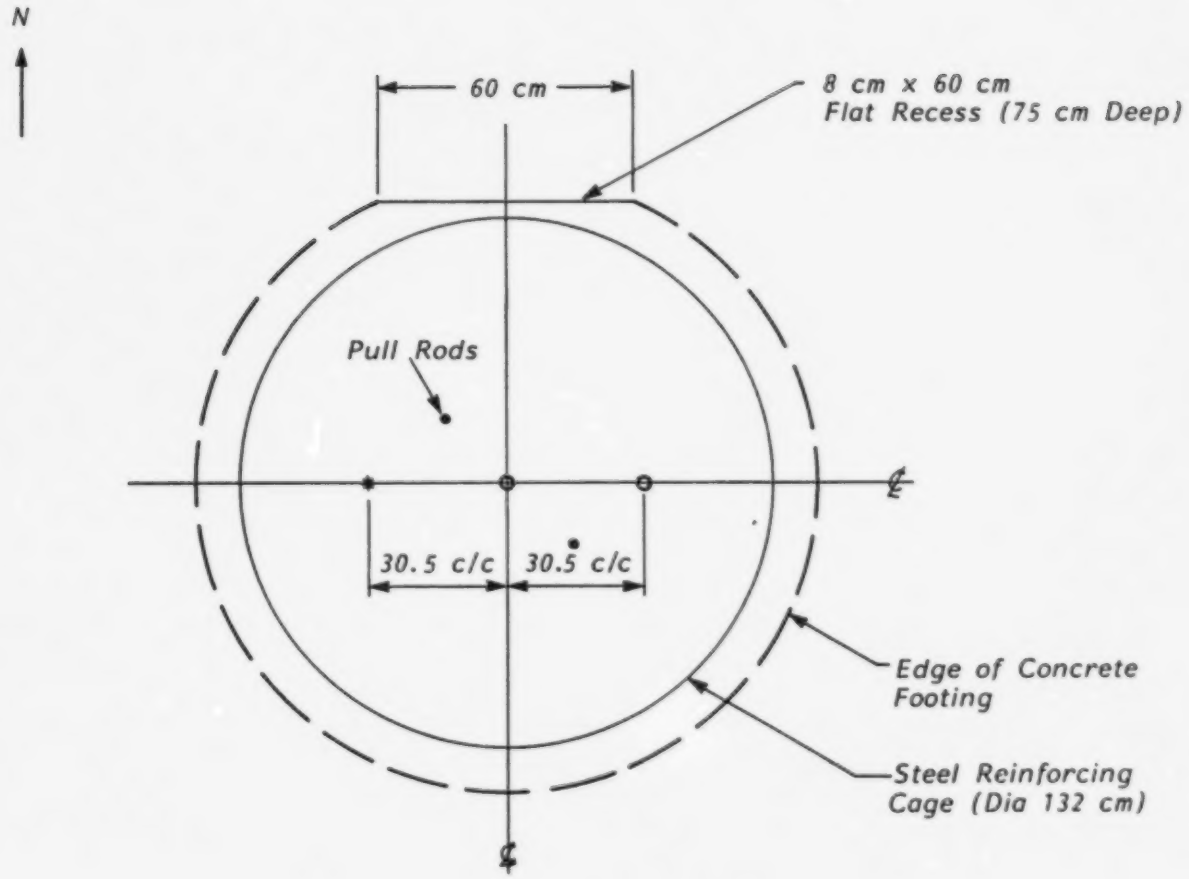
Note: Not to Scale (as Dimensioned in Centimeter from Top of Footing)

FIGURE 17B

INSTRUMENTATION OF TEST FOOTING #10

160 m Average Diameter x 6.71 m Embedded Length

Section A-A' - Plan View (+)



* IRAD Piezometer and Casing

o 2 PVC Access Tubes

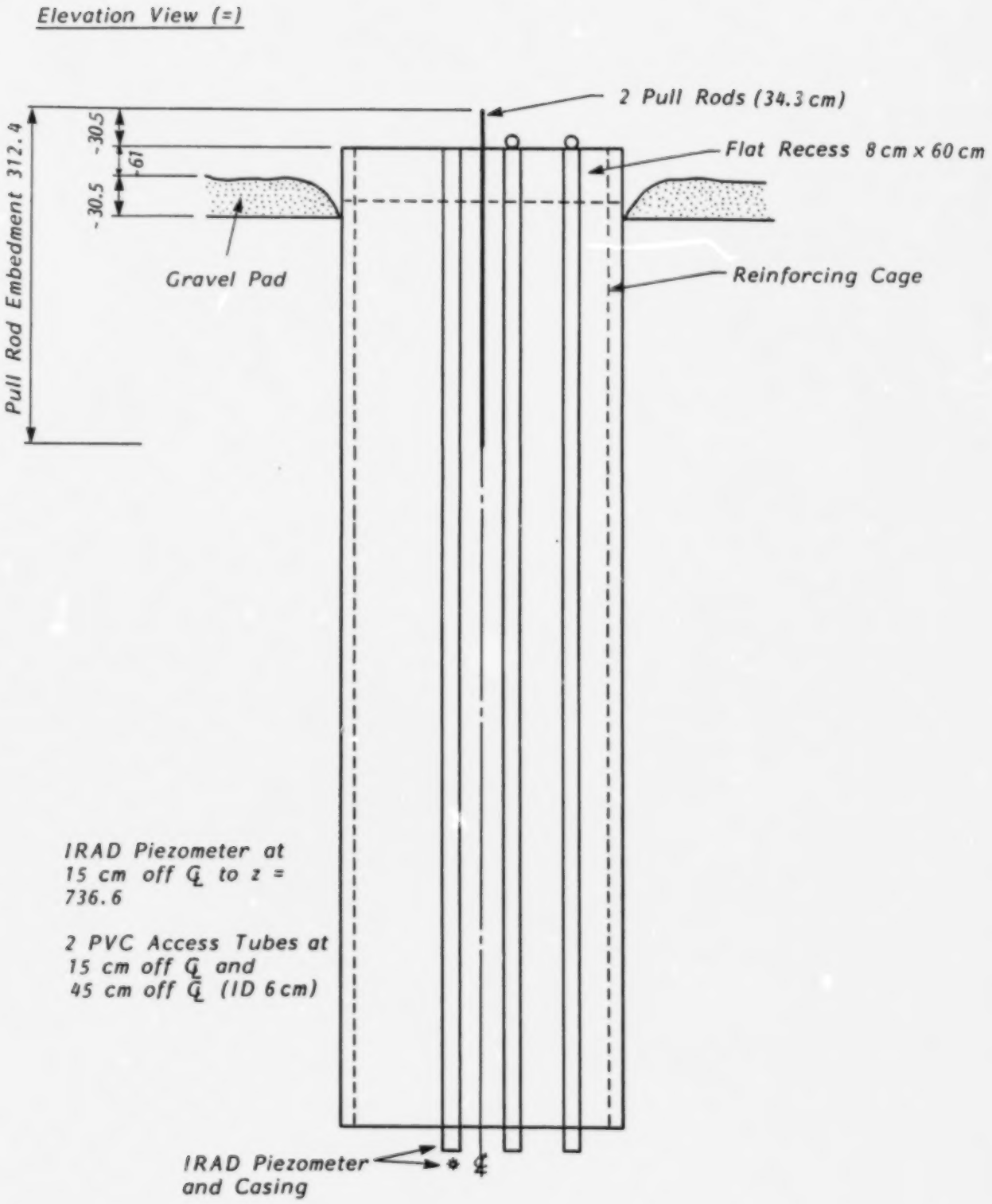
Note: (+) Not to Scale as Dimensioned

FIGURE 18B

INSTRUMENTATION OF THE TEST FOOTING #10

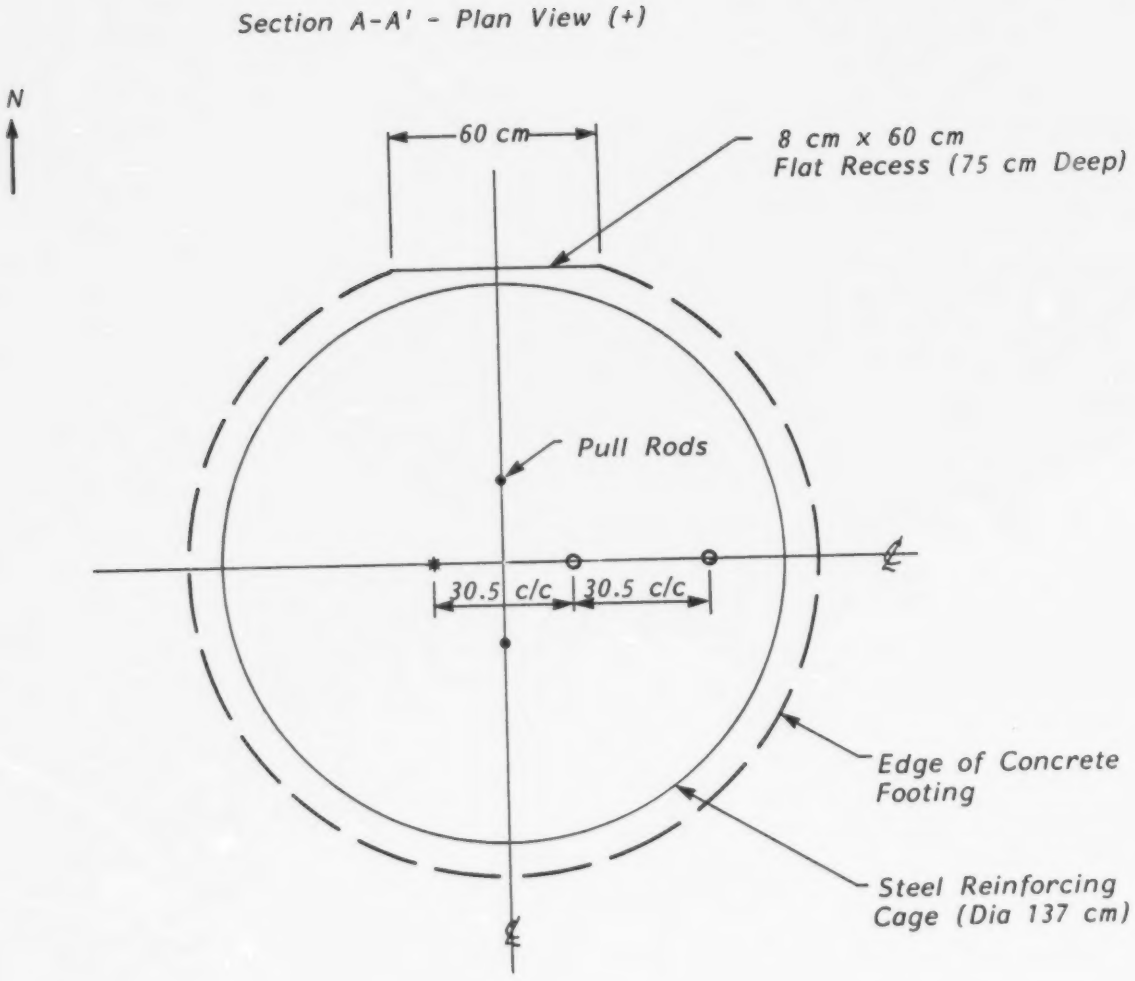
86-165

171328 RD



Note: Not to Scale (as Dimensioned in Centimeters from Top of Footing)

FIGURE 19B
INSTRUMENTATION OF TEST FOOTING #11
1.60 m Average Diameter x 6.65 m Embedded Length



* IRAD Piezometer and Casing

o 2 PVC Access Tubes

Note: (+) Not to Scale as Dimensioned

FIGURE 20B
INSTRUMENTATION OF THE TEST FOOTING #11

APPENDIX C

LOAD TEST DETAILS AND RESULTS

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TESTS ON CYLINDRICALLY AUGERED PIER FOUNDATIONS IN LEDA CLAY

G.A. Senkiw

A comprehensive test program consisting of uplift, compression, lateral load and overturning moment tests was conducted on full size reinforced concrete pier foundations to study their suitability for the proposed new 500 kV Transmission Line from Ottawa to Cornwall. These tests were carried out on a test site owned by the National Capital Commission south of Ottawa, Ontario. The tests commenced on October 3, and were completed on November 1, 1985.

Description of Footings, Tests and Test Procedures

A layout of the test site is given in Figure 1. A description of the footings in terms of diameter, installed depth and type of test performed is summarized in Table I and Figure 1.

i) Uplift Tests (Static and Cyclic)

The test arrangement for conducting a typical uplift test is shown in Figures 2 to 5. The uplift force on each footing was applied by two 2,700 kN (300 ton) capacity, hollow ram, hydraulic jacks which pulled on a pair of #18, Grade 60, Dywidag rebars which were set in the centre of the footing, 300 mm apart. The hydraulic jacks sat atop a thick steel bearing plate which had two predrilled holes to accommodate the two Dywidag pulling rebars. The bearing plate was placed on a pair of 48" WWF beams which rested on steel bearing pads set approximately 12 m (40 ft) apart. The uplift readings were continuously recorded by a linear voltage displacement transformer (LVDT). A dial gauge and a ruler with a transit were used for back-up. Both the LVDT and the dial gauge were attached to a batter board supported well away from the zone of influence. A load cell was used to measure the applied load.

In the static uplift tests the uplift force on each footing was applied in small increments of approximately 50 kN to permit readings of all instrumentation. At loads where the footings exhibited creep, the loads were maintained until the continuous creep subsided. Generally, the loading was continued until the footing started to uplift continuously with no further increase in load or the total uplift of footing was about 50 mm or more. In some cases, the loading was discontinued just at the failure load because the footing had to be used for other tests.

In the cyclic uplift test (Test No 2), the uplift force on the footing was applied in steps of approximately 100 kN up to a load of 660 kN. At that point the load was cycled 15 times from 190 kN to 660 kN in two steps. After the 15th cycle, the load was increased to 830 kN and then cycled for 5 cycles from 190 kN to 830 kN in two steps at which time the footing started to uplift rapidly.

ii) Compression Test

Figures 6 to 8 depict the test arrangement used for the compression test (Test No 3) on Footing #2 with Footings #1 and #3 for reaction. Two 2700 kN capacity, hollow ram, hydraulic jacks were used to apply the compression force to the centre footing. The jacks were set on a 100 mm thick by 1.2 m diameter steel plate placed on top of the centre footing (Footing #2). This steel plate distributed the load to the footing uniformly. Each jack was located directly under the web of each 48" WWF beam. A load cell was located between one jack and the thick loading plate to monitor the applied compression force on the footing (Figure 8). The beams, secured to each end footing (Footings #1 and #3) by Dywidag rebar and steel plates, were used as a reaction for the compression forces (Figure 7). Long timbers were used to prop the 18 m (60-foot) long WWF beams so that they would not topple during installation. Displacement readings of all three footings were recorded by dial gauges. The dial gauges were attached to batter boards which were supported well away from the footings. An LVDT and the load cell were used to obtain an autographic recording of the displacement versus load (Figure 8). The load cell reading was multiplied by 2 to establish the total applied compression load.

The loading sequence for the compression test was similar to the uplift tests. The foundation was loaded in increments of approximately 100 kN to permit recording of the various instruments. The test was terminated after settlement of the compression foundation reached 50 mm for the pending lateral load tests.

iii) Lateral Load Tests

The test arrangement used for the lateral load tests is illustrated in Figures 9 to 13. A horizontal force to both footings was applied simultaneously by one 900 kN (100 ton) capacity hydraulic jack. A steel reaction beam transferred the load to each footing. A load cell installed in line with the hydraulic jack recorded the applied

force. Both circular footings were poured such that their projections above ground level had parallel flats facing each other. A saddle plate with a 150 mm (6") diameter steel half round at each foundation served to maintain a horizontal alignment of the test equipment (Figures 12 and 13).

Horizontal displacements of each footing were measured with dial gauges attached to batter boards installed behind each footing. The batter boards were supported on timbers placed well away from the zone of influence. A LVDT with a load cell was used to obtain the load versus displacement curve for the test footing. The lateral forces were applied in increments of approximately 45 kN (10 kips). At each load increment, the dial gauges and all the rest of the instruments, ie, strain gauges, soil pressure gauges and inclinometers were read and recorded. The lateral loads were applied until the horizontal displacement of the test footing exceeded 50 mm (2") or the limit of the jack travel was reached.

iv) Combination Lateral and Uplift Load Test

One combination lateral and uplift load test (Test No 5) was performed. Figures 14 to 16 depict the test arrangement. Basically, the test arrangement incorporated the same equipment used for uplift and lateral load tests. In this test the uplift reaction beams were spaced further apart to permit some tilting of the pulling rods due to the applied lateral load to the test footing. Two sets of hydraulic jacks, pumps, dial gauges, load cells and LVDT's were utilized for this test. Two separate records of load vs displacement were obtained for the lateral and uplift behaviour of the test footing under combined loading.

The test was conducted by first applying the lateral load followed by twice the uplift load at each increment. Thus, the lateral loads were applied in 45 kN increments while the uplift loads were applied in 90 kN increments. This loading program was followed until the lateral load reached 180 kN (40 kips). At that point, the lateral load was maintained while the uplift load was incrementally increased. The uplift load was increased until the footing uplift exceeded 50 mm at which point the test was terminated.

v) Long Term Uplift Test

Figures 17 to 19 illustrate the long term uplift test arrangement (Test No 9). The basic test setup was similar to the static uplift test. The uplift force to the footing was applied by two 900 kN (100 ton) capacity, hollow ram, hydraulic jacks which pulled on a pair of #18, Grade 60, Dywidag rebars which were previously cast into the centre of the footing. The jacks rested on a thick steel bearing plate which had two through holes and sat on top of a pair of 36" WWF beams. The 9 m long WWF beams were placed on top of large wood timber bearing pads. The wood bearing pads were placed approximately 6 m apart. A dial gauge was used to record the footing uplift movements. It was attached to a batter board which was supported on two steel stakes installed well away from the zone of influence.

The uplift loads were applied in steps of 90 kN up to a load of 360 kN, after which the steps were decreased to 45 kN until a load of 535 kN was reached. At each step, uplift readings and pore pressure changes were recorded. After the 535 kN load was reached it was maintained for two days. The loads were maintained by an air-operated Sprague pump which requires a constant air pressure. A nitrogen tank was used to supply the constant pressure. A hydraulic pressure gauge connected in line with the pressure hoses to the hydraulic jacks monitored the uplift load applied to the footing. On the first day, uplift readings were taken every hour and on the following day only early in the morning at noon and at quitting time. On the third day the load was increased to 690 kN after which the load was lowered to 580 kN and maintained until the hydraulic jacks ran out of travel and the footing failed in uplift. At that point the test was terminated.

vi) Overturning Moment Tests

The overturning moment tests utilized three similar test arrangements. In total four tests were performed on three footings. For each test two footings were utilized by pulling them towards each other at the top and applying a restraining load at the ground level. In this way, a predetermined overturning moment to shear ratio was maintained during the test.

a) Test No 10 (Footing #7 against Footing #8)

Figures 20 to 23 show the test arrangement for the overturning moment test involving the 1.60 m diameter footing #7 against the 2.76 m diameter footing #8. No 18 Dywidag rebars together with a 2700 kN hollow ram hydraulic jack were used to apply the moment force to both footings simultaneously at the 3.05 m level. A lateral restraint force just above ground level on both footings was applied by a steel beam together with a 900 kN hydraulic jack. A load cell at each load application point recorded the applied loads. Dial gauges attached to batter boards installed behind each footing monitored the horizontal displacement of each footing near the ground level.

The upper loads to both footings were applied in steps of 45 kN while the bottom loads were maintained to be 90 percent of the upper loads. This simulated a resultant moment arm of approximately 30 m. An autographic record of the top load versus lateral displacement near the ground level of Footing #7 was obtained. At each load increment all the instruments were read and recorded. The footings were loaded until the top load reached the yield strength of the Dywidag pulling rods. At that point the top and bottom lateral loads were unloaded in 3 or 4 steps to zero. Before the end of the day, the overturning loads were reapplied but only to the 105 kN load level. For this test only the lateral displacement of Footing #7 was recorded. The following day, only the top loads were reapplied with no lateral restraint bracing, ie, the hydraulic jack was removed to allow the footings to move freely at the ground level. The test was terminated once the horizontal displacement of Footing #7 near the ground level exceeded 65 mm.

b) Test No 11 (Footing #6 against Footing #8)

The test arrangement for Test No 11 is depicted in Figures 24 to 26. Although two sets of 3-1/2" diameter pulling rods and 2700 kN hollow ram hydraulic jacks were installed only the top set of rods together with the top jack and load cell were actually used. The bottom lateral restraint force was provided by a 20" diameter steel pipe together with an 8900 kN (1,000 ton) capacity hydraulic jack. As in Test No 10, dial gauges recorded the lateral displacements of each footing near ground level.

The top load to both footings was increased in steps of 200 kN while the bottom load was kept at 90 per cent of the top load. The same sequence of readings of all the instrumentation was carried out in this test as in the previous overturning test. An autographic record of the top load versus the horizontal displacement of Footing #6 near the ground was obtained. Once the top load of 2250 kN and the bottom of 2065 kN was reached an attempt was made to remove both loads incrementally and reapply the loads with both sets of pulling rods and hydraulic jacks, but due to a hydraulic pump failure the test was discontinued.

c) Test No 12 (Footing #8 versus Footings #7, 6 and 9, 10, 11)

Test No 12 involved overturning and horizontally displacing Footing #8 using Footing #7 and Footing #6. Footing #6 was tied to Footings #9, 10 and 11 as shown in Figures 27 to 30. This was to provide additional reaction for pushing over Footing #8. Both sets of 3-1/2" diameter pulling rods together with hydraulic jacks were used between Footings #6 and #8 to apply the top loads. No lateral restraint force at the ground level was employed between these two footings. However, a lateral load was applied to Footing #8 at the ground level from behind by reacting against Footing #7 as shown in Figure 31.

The first part of the overturning moment test involved applying a lateral load to Footing #8 at the ground line by means of a strut and a jack reacting against Footing #7. This load was applied in steps of 90 kN up to a maximum of 355 kN. At that point the upper loads between Footing #6 and #8 were increased in steps of approximately 200 kN. These top loads were increased until the horizontal displacement of Footing #8 at the ground level reached 25 mm. During this stage the ground line force between Footing #7 and #8 dropped to 100 kN. Once this horizontal displacement was achieved, the loads were held for 17 minutes. After the hold, one final top load increment was applied after which the test was subsequently terminated.

Test Results

A summary of all the test results is presented in Table I. Details of all these tests are given in Tables II to XIV and are also shown in Figures 32 to 45.

Autographic records that were obtained during these tests are not included in this report. They were used to guide the loading sequence and hold periods of loads by watching the rate of creep movement during the test.

Certain tests were discontinued short of complete failure because the footings had to be reused for other tests. There were only two tests which could not be continued due to equipment problems or limitations. In Test No 11, the small hydraulic pump which powered the 1,000 ton jack, due to oil temperature rise and limited reservoir size, failed when attempting to release the pressure. In the future, the large capacity, air-cooled, heavy-duty pump should be used. In Test No 12, larger overturning moment loads could not be applied because the Dywidag tie-back system was not designed for accommodating higher lateral loads. In the future, if high lateral loads are anticipated, a high capacity lateral tie-back system must be designed.

Approved:

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Submitted:

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GAS:sf

TABLE I

SUMMARY OF TESTS AND TEST RESULTS

Test No	Test Date	Test Type	Footing No	Footing Dimensions Dia (m)	Footing Dimensions Depth (m)	Maximum Recorded Load (kN)	Maximum Recorded Displacement (mm)		Detailed Record Table No	Detailed Record Figure No
1	Oct 3/85	Uplift	9	1.60	6.88	970	21.84		II	32
2	Oct 22/85	Cyclic Uplift	11	1.60	6.65	830	14.91		III	33
3	Oct 23/85	Compression	2	1.50	6.40	595	>50		IV	34
4	Oct 23/85	Uplift	1	0.92	9.65	445	49.28		V	35
5	Oct 24/85	Combined Lateral and Uplift	3	1.50	6.40	700 180	>50 15.27	Uplift Lateral	VI VI	36 37
6	Oct 25/85	Lateral	2 3	1.50 1.50	6.40 6.40	580 580	41.40 63.63		VII VII	38
7	Oct 25/85	Uplift	10	1.60	6.71	655	34.42		VIII	39
8	Oct 28/85	Lateral	1 2	0.92 1.50	9.65 6.40	425 425	59.69 35.69		IX	40
9	Oct 28/85	Long Term Uplift	4	1.50	6.81	690	>44		X	41
10a	Oct 29/85	Overturning Moment	7 8	1.60 2.76	6.93 12.73	1155 1155	38.25 2.87		XI	42
10b	Oct 29/85	Overturning Moment	7 8	1.60 2.76	6.93 12.73	625 625	65.46 6.27		XII	43
11	Oct 31/85	Overturning Moment	6 8	2.49 2.76	6.63 12.73	200 200	25.58 5.64		XIII	44
12	Oct 31/85	Overturning Moment (with Footings #7, 9, 10, 11 for lateral restraint)	6 8	2.49 2.76	6.63 12.73	355 355	35.13 37.85		XIV	45

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TABLE II

UPLIFT TEST ON FOOTING #9

Footing Diameter: 1.60 m
Installed Depth: 6.88 m

Load (kN)	Uplift, mm				
	Start	1 min	5 min	10 min	15 min
0	0				
80	0	-	0.20		
115	0.23	0.23	0.25		
170	0.38	0.38	0.46		
215	0.56	0.56	0.56		
280	0.71	0.71	0.76		
320	0.89	0.89	0.89		
375	1.12	1.12	1.14		
405	1.35	1.35	1.35		
490	1.57	1.57	1.57		
555	1.80	1.80	1.80		
615	2.03	2.03	2.03		
670	2.26	2.26	2.29		
730	2.57	2.59	2.59		
775	2.82	2.87	2.92		
820	3.25	3.43	3.53		
870	4.14	4.39	5.03	6.73	7.80
925	8.64	10.29	13.97	16.00	17.78
970	18.29	21.84	———	quick uplift	

TABLE III

TEST NO 2
CYCLIC UPLIFT TEST ON FOOTING #11

Footing Diameter: 1.60 m
Installed Depth: 6.65 m

Load (kN)	Uplift (mm)						
0	0						
160	0.33						
250	0.56						
355	0.81						
455	1.09						
560	1.42						
660	1.83	Cycle #1					
190	0.81			0.79		0.79	
355	1.12			1.14		1.22	
665	1.98		#2	1.96	#3	2.06	#4
190	0.86			0.89		0.89	
355	1.24			1.30		1.27	
665	2.11		#5	2.11	#6	2.08	#7
190	0.89			0.91		0.89	
355	1.30			1.30		1.27	
665	2.08		#8	2.13	#9	2.13	#10
190	0.91			0.94		0.97	
355	1.35			1.32		1.35	
665	2.16		#11	2.16	#12	2.21	#13
190	0.99			1.04			
355	1.37			1.42			
665	2.21		#14	2.31	#15		
720				2.46			
775				2.64			
830				2.97			
190	1.37			1.88		2.21	
470	2.08			2.59		2.90	
830	3.56		#1	4.09	#2	4.47	#3
					5 min	10 min	
190	2.62			3.00			
470	3.28			3.68			
830	4.88	(4)		5.28	7.39	14.91	sud'enly crept out

TABLE IV

TEST NO 3
COMPRESSION TEST ON FOOTING #2

Footing Diameter: 1.50 m
Installed Depth: 6.40 m

Displacement, mm																						
Load (kN)	Downward Displacement, Footing #2						Uplift, Footing #7						Uplift, Footing #3					
	Hold Time, minutes						Hold Time, minutes						Hold Time, minutes					
	Start	5	10	15	20	30	Start	5	10	15	20	30	Start	5	10	15	20	30
0	0						0						0					
180	4.06						0.10						0					
240	4.62						0.13						0.10					
330	5.41						0.25						0.15					
420	6.73	9.53	10.59	11.35	11.99	13.03	0.38	0.33	0.28	0.22	0.20	0.15	0.31	0.33	0.33	0.28	0.23	0.10
435	13.59	14.62	(3 minutes)				0.13	0.08	(3 minutes)				0.10	0.03	(3 minutes)			
445	14.43	15.24					0.08	0.03					0.03	-0.05				
465	16.05	16.38	(2 minutes)				0.03	0	(2 minutes)				-0.05	-0.05	(2 minutes)			
490	17.30			21.06			-0.05			-0.10			-0.05			-0.31		
515	22.73			30.18			-0.10			-0.03			-0.31			-0.31		
545	32.64			43.31			-0.03			-0.10			-0.31			-0.31		
556	49.28						-0.13						-0.31					
595	>50						-0.03											

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TABLE V

TEST NO 4
UPLIFT TEST ON FOOTING #1

Footing Diameter: 0.92 m
Installed Depth: 9.65 m

Load (kN)	Uplift, mm				
	Start	5 min	10 min	20 min	30 min
0	0				
55	0.25				
105	0.43				
185	0.94				
215	1.14				
260	1.58				
305	1.98				
345	2.52				
390	3.30				
445	5.46	9.78	14.86	27.69	49.28

TABLE VI

TEST NO 5
COMBINED LATERAL AND UPLIFT LOADS ON FOOTING #3

Footing Diameter: 1.50 m
Installed Depth: 6.40 m

Lateral (Ftg #2-3)	Uplift (Ftg #3)		Displacement, mm		
			Footing #2	Footing #3	
			Lateral	Lateral	Uplift
0	0		0	0	0
40	60		0.31	0.13	0.13
75	160		0.79	0.79	0.76
140	265		1.60	1.50	1.27
180	355		2.36	2.64	1.73
170	465		2.57	2.69	2.49
175	565		2.59	2.92	3.30
155	675	Start	2.64	3.02	4.70
165	650	7 minutes	2.82	4.09	6.99
170	650	15 minutes	2.90	5.06	9.37
160	700	Start	2.90	5.56	10.92
165	695	5 minutes	2.90	7.60	19.94
165	695	10 minutes	2.92	8.79	27.56
165	680	15 minutes	2.95	10.95	37.34
165	635	20 minutes	3.00	15.27	>50
0	0		1.09		

TABLE VII

TEST NO 6
LATERAL LOAD TEST BETWEEN FOOTING #2 AND #3

Load (kN)		Horizontal Displacement, mm	
		Footing #2	Footing #3
		Footing Diameter: 1.50 m Installed Depth: 6.40 m	Footing Diameter: 1.50 m Installed Depth: 6.40 m
0		0	0
45		0.36	0.36
80	18 minute hold	0.79 -> 0.66	1.09 -> 1.93
135		1.07	2.82
175	11 minute hold	1.75 -> 2.26	4.14 -> 5.54
215		2.77	6.43
260	15 minute hold	3.81 -> 4.60	8.31 -> 11.05
295		5.18	12.09
345	10 minute hold	6.30 -> 7.98	14.55 -> 17.37
380		8.87	18.75
425	10 and 20 minute hold	10.74 -> 13.06 -> 13.95	22.30 -> 26.06 -> 27.03
450		14.58	28.14
505	10 minute hold	18.01 -> 25.32	33.73 -> 42.62
55		26.90	43.89
580	10 minute hold	32.49 -> 41.63	50.24 -> 61.16
510	1 hr and 48 minute hold	41.40	63.70
345		39.60	63.63
170		35.28	59.13
0		23.34	45.03

TABLE VIII

TEST NO 7
UPLIFT TEST ON FOOTING #10

Footing Diameter: 1.60 m
Installed Depth: 6.71 m

Load (kN)	Displacement, mm Start
0	0
70	0.38
165	0.89
245	1.35
345	1.91
470	2.72
515	2.92 -> 3.12 (3 minutes)
565	3.56
615	4.04
655	4.83 -> 34.42 (5 minutes)
0	4.32

TABLE IX

TEST NO 8
LATERAL LOAD TEST BETWEEN FOOTING #1 AND #2

Load (kN)		Horizontal Displacement, mm Footing #1 Footing Diameter: 0.92 m Installed Depth: 9.65 m	Footing #2 Footing Diameter: 1.50 m Installed Depth: 6.40 m
0		0	0
40		0.69	0.76
75	Start	1.60	1.78
	10 minute hold	2.11	2.64
130		3.40	4.34
155		4.75	6.15
200		7.37	9.53
240	Start	13.21	12.07
	10 minute hold	16.84	14.17
285		21.08	16.13
335	Start	27.69	19.18
	7 minute hold	33.73	21.34
370	Start	37.08	24.38
	7 minute hold	42.42	25.53
425	Start	59.69	30.73
355	1 hour and 35 minute hold	-	35.69
284			34.80
150			28.96
0			6.10

TABLE X

TEST NO 9
LONG TERM UPLIFT TEST ON FOOTING #4

Footing Diameter: 1.50 m
Installed Depth: 6.81 m

Load (kN)	Date and Time		Uplift (mm)
0	Oct 28	2:45 pm	0
95			0.25
190			0.45
285			0.85
360			1.25
400			1.38
460			1.71
495			1.95
535		3:00 pm	2.23
475		3:12 pm	2.15
535		3:13 pm	1.36
535		3:30 m	2.60
535		4:00 pm	2.73
	Oct 29	8:45 am	4.65
		11:30 am	4.65
		5:30 pm	5.21
	Oct 31	8:00 am	5:31
535		8:03	5.47
570		8:05	6.15
630		8:06	7.55
665		8:08	10.40
690		8:10	10.53
630		8:25	11.71
600		8:40	12.01
575		8:45	13.65
610		9:10	13.99
580		10:10	19.85
580		10:40	23.80
590		11:30 am	27.65
600		1:30 pm	30.75
570		3:00 pm	37.25
570		5:00	42.00
570			
out of jack travel	Nov 1	8:15 am	7.44

TABLE XI

TEST NO 10a
OVERTURNING MOMENT TEST BETWEEN FOOTINGS #7 AND #8

Resultant Shear Load (kN)		Resultant Moment kN m	Eccentricities (m)	Horizontal Displacement, mm	
				Footing #7	Footing #8
				Footing Diameter: 1.60 m	Footing Diameter: 2.76 m
				Installed Depth: 6.93 m	Installed Depth: 12.73 m
0		0		0	0
5		145	29	0.69	0.20
10		240	24	0.97	0.31
15		360	24	1.07	0.46
15		480	32	1.35	0.53
20		600	30	1.47	0.53
25		725	29	1.50	0.53
30		845	28	1.12	0.51
30		965	32	1.45	0.51
40		1085	27	1.63	0.56
45		1205	27	2.01	0.69
45		1325	29	2.08	0.66
50		1450	29	2.31	0.71
55		1685	30	3.18	0.91
60		1925	32	4.50	1.12
75		2170	29	5.51	1.17
105		2410	24	7.26	1.40
150		2545	17	7.85	1.47
195	Start	2665	14	10.95	2.03
	13 minutes			15.57	2.18
285	Start	2920	10	22.63	2.82
	13 minutes			25.32	2.85
375		3175	8.5	26.24	2.87
375	(unloading)	3175	8.5	26.70	2.85
105	(unloading)	2410	24	25.63	-
45	(unloading)	1325	27	22.63	-
0		0	0	12.42	1.50
45				25.65	-
105				38.25	-
45				31.50	-
0				20.22	-

TABLE XII

TEST NO 10b (Bottom Jack removed)
OVERTURNING MOMENT TEST BETWEEN FOOTINGS #7 AND #8

Resultant & Shear Load (kN)		Resultant Moment* (kN m)	Horizontal Displacement, mm Footing #7 Footing Diameter: 1.60 m Installed Depth: 6.93 m	Footing #8 Footing Diameter: 2.76 m Installed Depth: 12.73 m
0		0	0	0
90		255	1.30	0.23
180		515	5.13	0.69
265		755	9.75	1.14
310	Start	885	12.40	1.40
	4 minutes	1010	13.13	-
355	Start		15.60	1.80
	7 minutes	1140	16.84	-
400	Start		18.36	2.06
	12 minutes	1270	19.96	-
445	Start		21.39	2.41
	6 minutes	1575	22.66	-
535	Start		29.64	3.28
	15 minutes	1655	39.68	3.76
580	Start		44.50	4.34
	15 minutes	1780	60.27	4.93
625	Start		65.46	5.33
	15 minutes		-	5.97
	30 minutes		-	6.27
415		1180	-	5.79
265		755	-	4.93
90		255	-	3.71
0			-	2.29

For resultant moment multiply the resultant shear load by moment arm of 2.85 m.

TABLE XIII

TEST NO 11
OVERTURNING MOMENT TEST BETWEEN FOOTINGS #6 AND #8
(with top and bottom jacks)

Resultant Shear Load (kN)	Resultant Moment (kN m)	Horizontal Displacement, mm Footing #6 Footing Diameter: 2.49 m Installed Depth: 6.63 m	Horizontal Displacement, mm Footing #8 Footing Diameter: 2.76 m Installed Depth: 12.73 m
0	0	0	0
40	525	1.27	0.89
40	940	2.74	1.96
75	1460	3.10	2.11
110	1985	4.17	2.46
110	2385	4.85	2.46
135	2790	7.90	3.30
170	3375	8.51	3.43
165	3570	9.65	4.22
180	3815	10.54	4.39
165	3990	11.02	4.29
180	4235	11.56	4.37
180	4440	15.90	5.64
200	4710	20.14	5.13
155	4905	20.83	4.93
200	5130	22.84	5.36
180	5330	25.58	5.33

$\bar{e}$ (average eccentricity) = 22.90 m

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TABLE XIV

TEST NO 12
OVERTURNING MOMENT TEST BETWEEN FOOTINGS #6 AND #8
WITH TOP JACKS ONLY

Lateral Load P_2 From Ftg #7* (kN)	Top Load P_1 (kN)	Tie-Back** Load to Ftg #9,10,11 (kN)	Resultant Load on Footing #8 Shear (kN)	Resultant Load on Footing #8 Moment (kN m)	Horizontal Displacement, mm Footing #6 Footing Diameter: 2.49 m Installed Depth: 6.63 m	Horizontal Displacement, mm Footing #8 Footing Diameter: 2.76 m Installed Depth: 12.73 m
0	0	0	0	0	0	0
90	0	100	90	20		0.36
180	0	100	180	35		1.02
265	0	100	265	55		1.63
355	0	100	355	70		2.62
355	215	185	560	635	6.99	3.76
310	465	455	775	1285	17.81	6.35
260	590	630	850	1605	26.87	13.59
190 Start	750	800	940	2010	35.13	16.64
265 (17 min hold)	750	800	1015	2025	-	23.37
100	1120	1210	1220	2965	-	37.85
0	0	-			27.51	23.88

* Additional Lateral Load was applied to Footing #8 at ground level reacting against Footing #7.

** Footing #6, which acted as a Reaction, was tied to Footings #9, 10 and 11 at ground level (see Figure 31).

Resultant moment = $P_1e_1 + P_2e_2$

$e_1 = 2.63$ m

$e_2 = 0.20$ m

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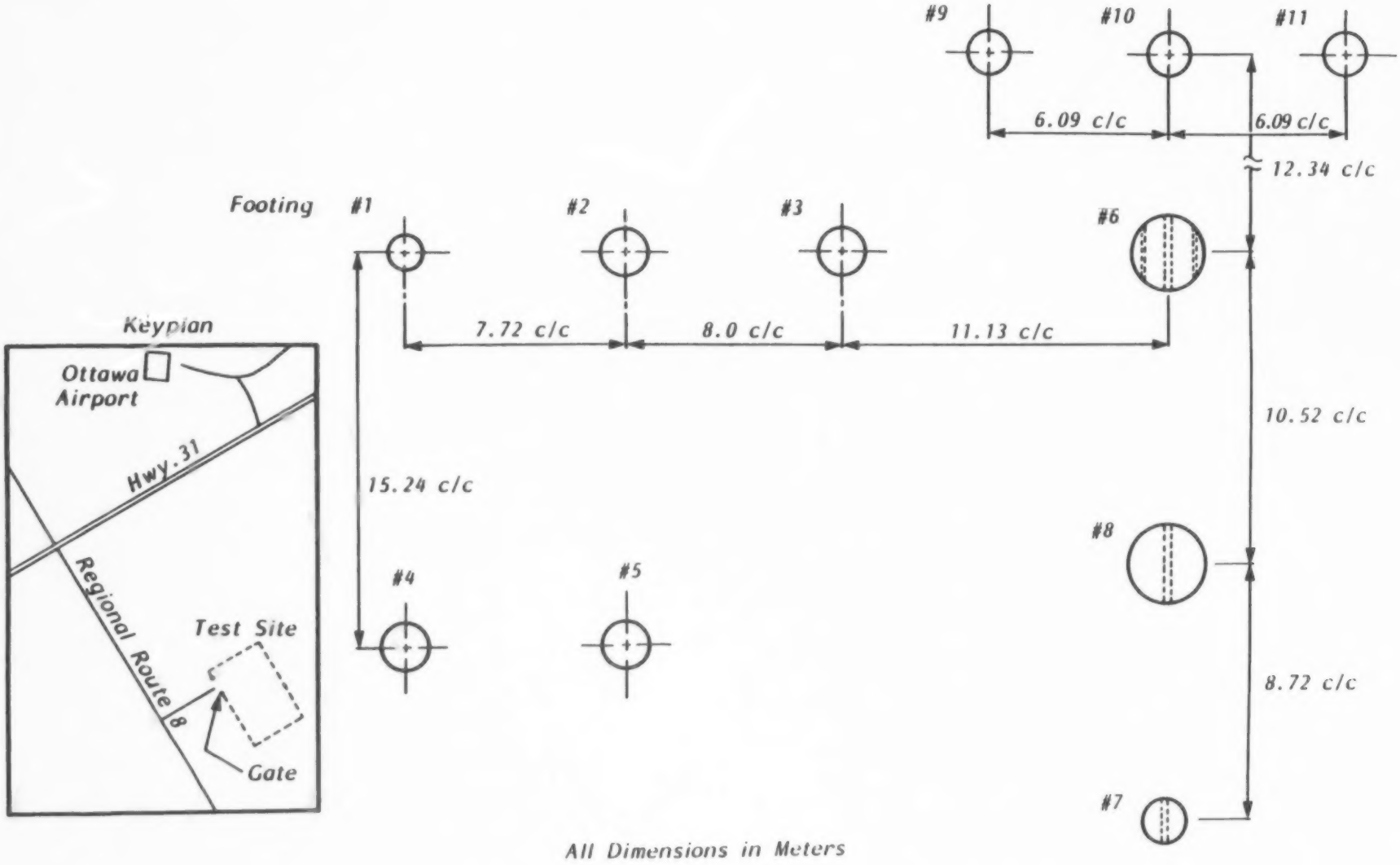

FIGURE 1

LAYOUT OF FOUNDATION FOOTINGS
AT GLOUCESTER TEST SITE

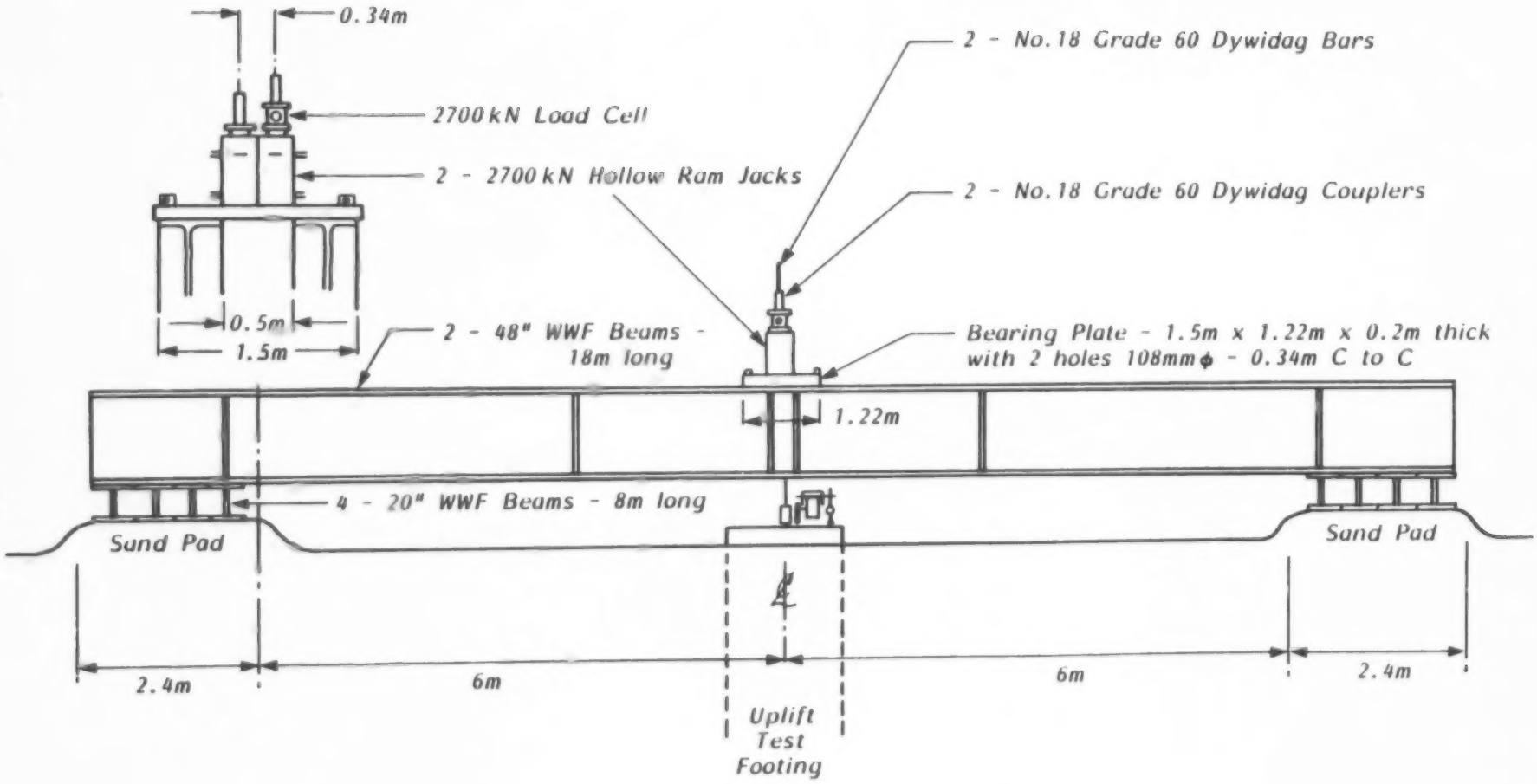
0.34m
2700kN Load Cell
2 - 2700kN Hollow Ram Jacks
2 - No.18 Grade 60 Dywidag Bars
2 - No.18 Grade 60 Dywidag Couplers
0.5m
1.5m
2 - 48" WWF Beams - 18m long
Bearing Plate - 1.5m x 1.22m x 0.2m thick with 2 holes 108mmϕ - 0.34m C to C
1.22m
4 - 20" WWF Beams - 8m long
Sand Pad
Sand Pad
2.4m
6m
6m
2.4m
Uplift Test Footing

FIGURE 2
UPLIFT TEST ARRANGEMENT

FIGURE 3

STATIC UPLIFT TEST ARRANGEMENT

FIGURE 4

CLOSE-UP OF BEARING PLATE, HYDRAULIC JACKS, LOAD CELL AND DYWIDAG PULLING BARS

FIGURE 5

INSTRUMENTATION FOR MONITORING FOOTING UPLIFT

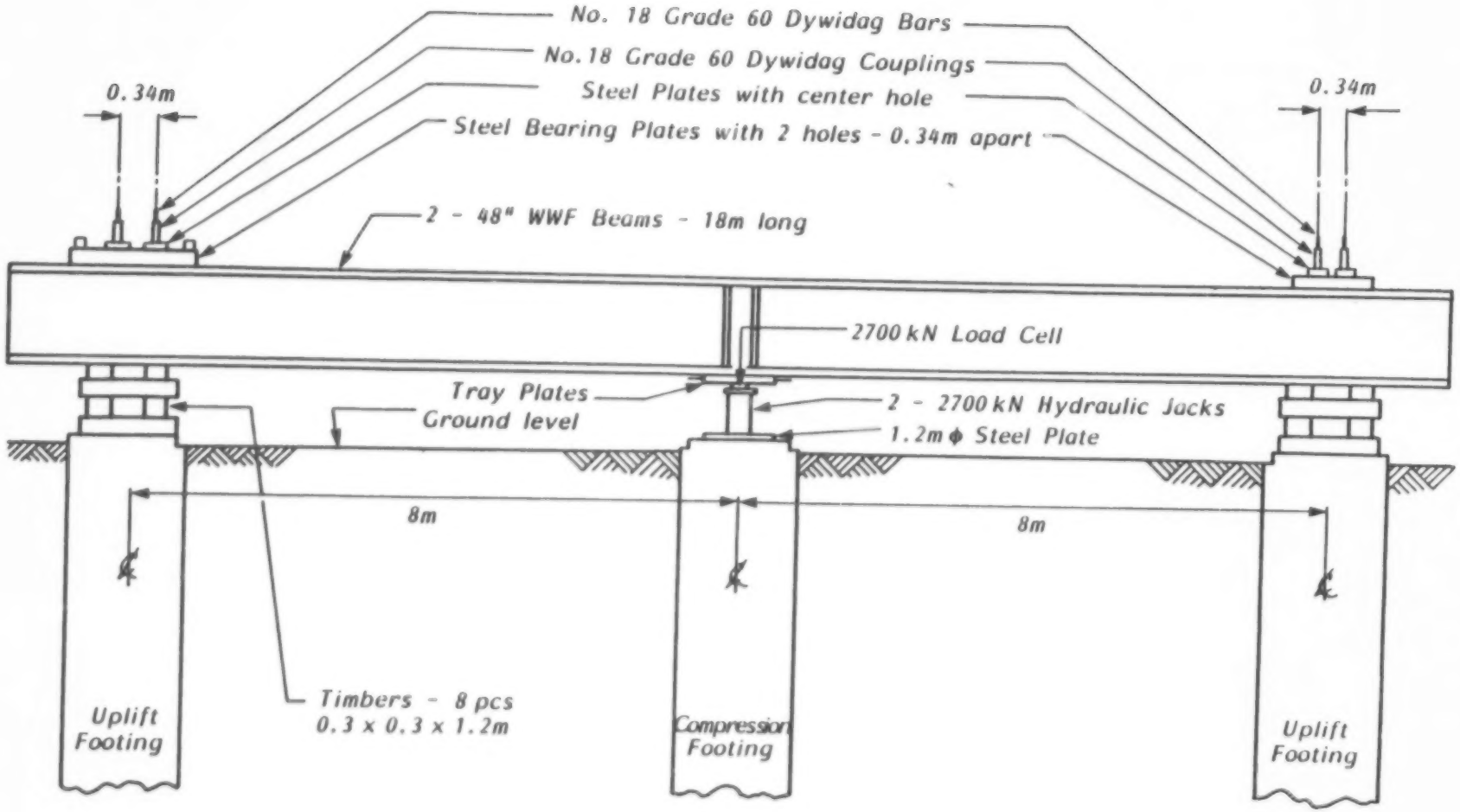

FIGURE 6
COMPRESSION TEST ARRANGEMENT

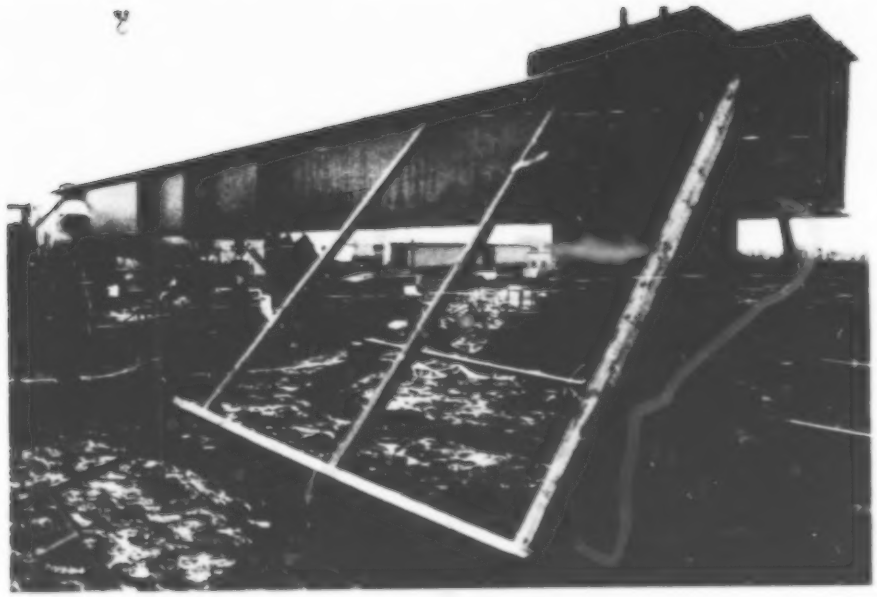

FIGURE 7
COMPRESSION TEST ARRANGEMENT

FIGURE 8
CLOSE-UP OF THE HYDRAULIC JACKS AND INSTRUMENTATION

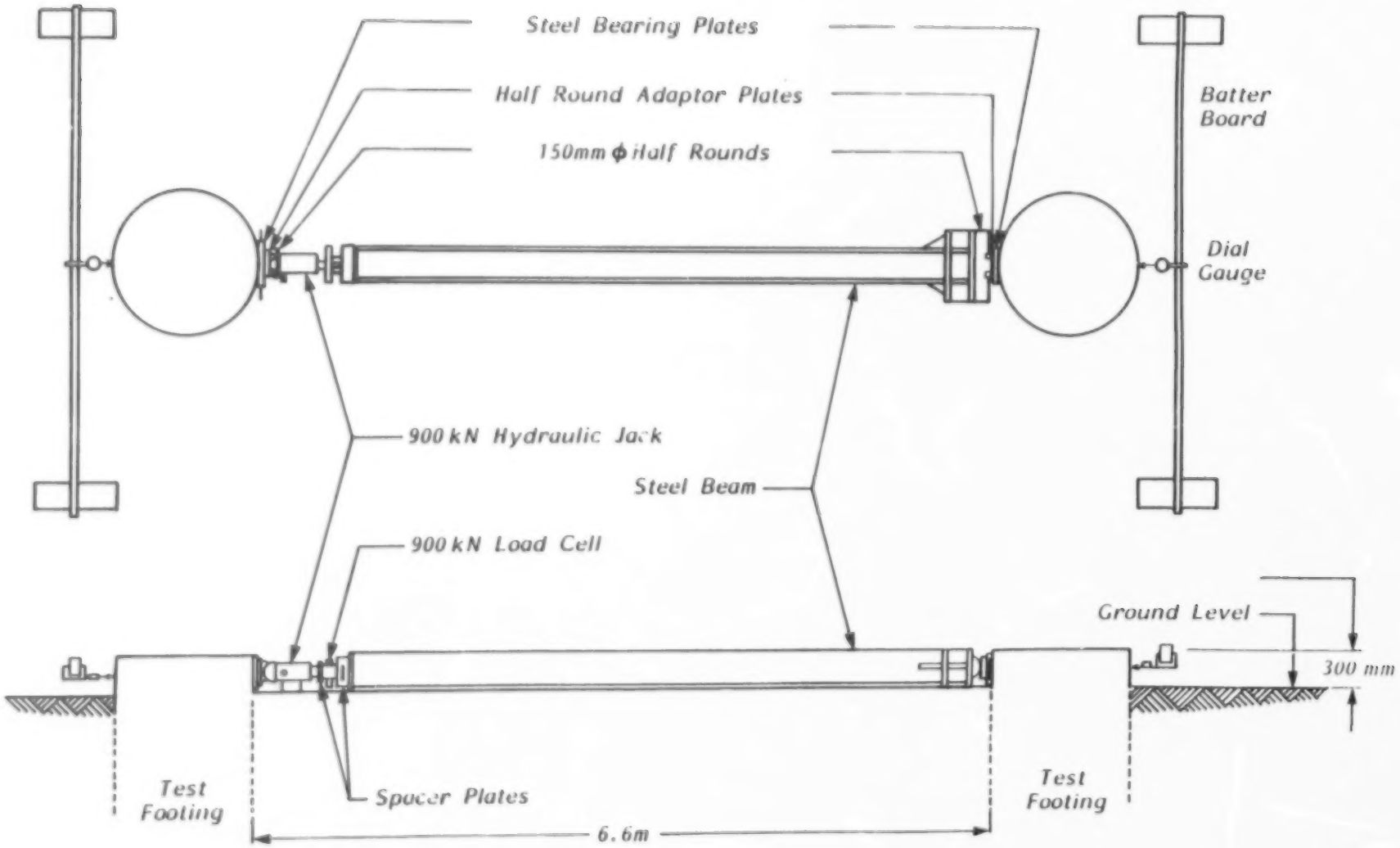

FIGURE 9
LATERAL LOAD TEST ARRANGEMENT

FIGURE 10
LATERAL LOAD TEST ARRANGEMENT

FIGURE 11
HYDRAULICS AND AUTOGRAPHIC
RECORDING INSTRUMENTATION

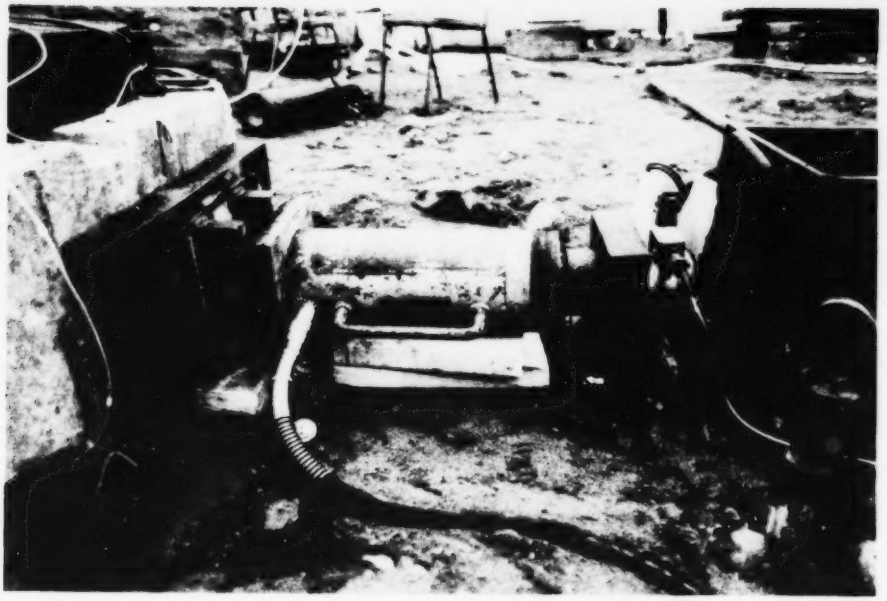

FIGURE 12

CLOSE-UP OF STEEL BEARING PLATES
HYDRAULIC JACK AND LOAD CELL

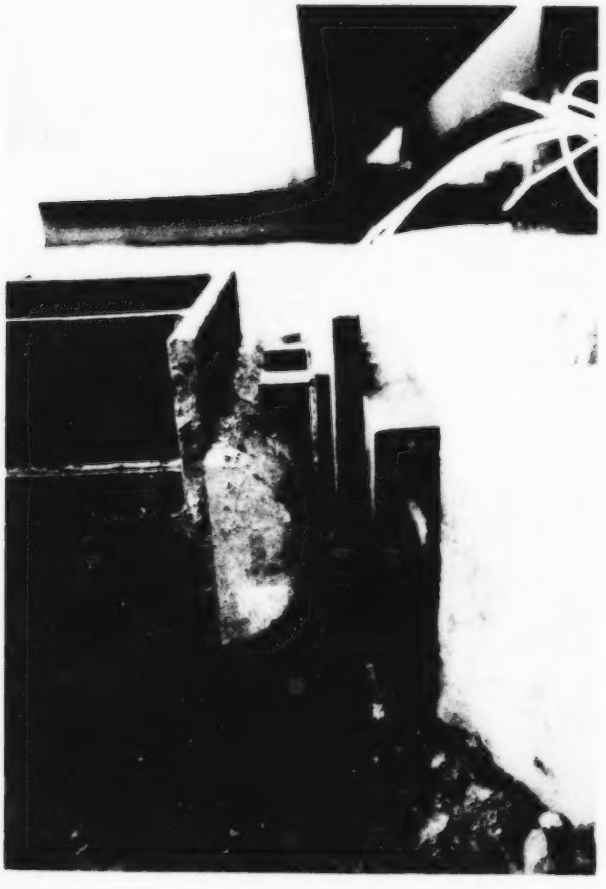

FIGURE 13

CLOSE-UP OF THE
OTHER FOOTING
REACTION

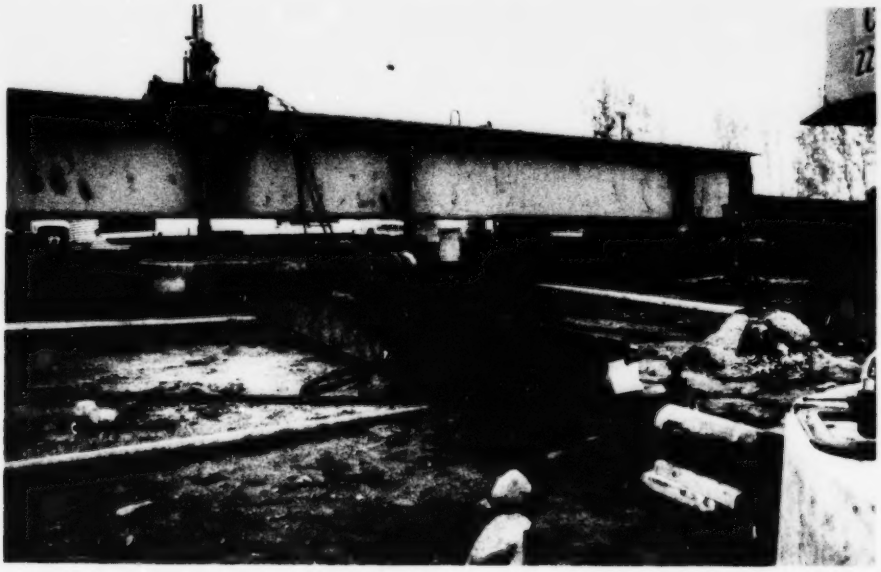

FIGURE 14
COMBINED LATERAL LOAD PLUS UPLIFT TEST

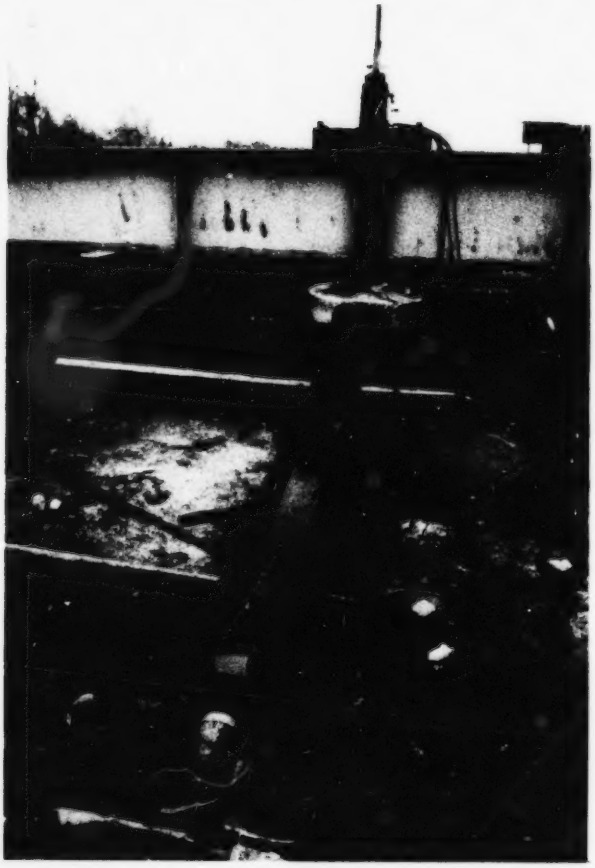

FIGURE 15
ANOTHER VIEW OF COMBINED
LATERAL LOAD PLUS UPLIFT TEST

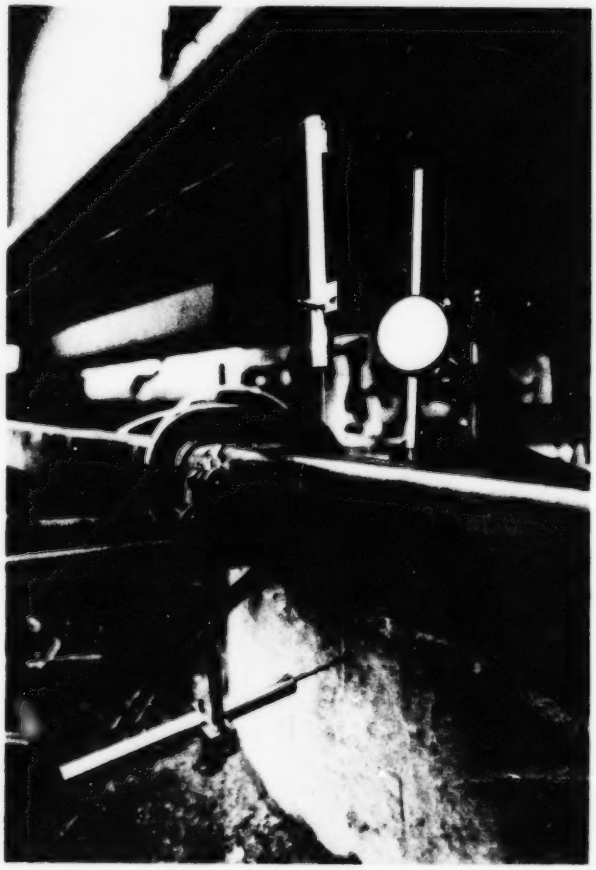

FIGURE 16
INSTRUMENTATION FOR MEASURING
LATERAL LOAD PLUS UPLIFT TEST

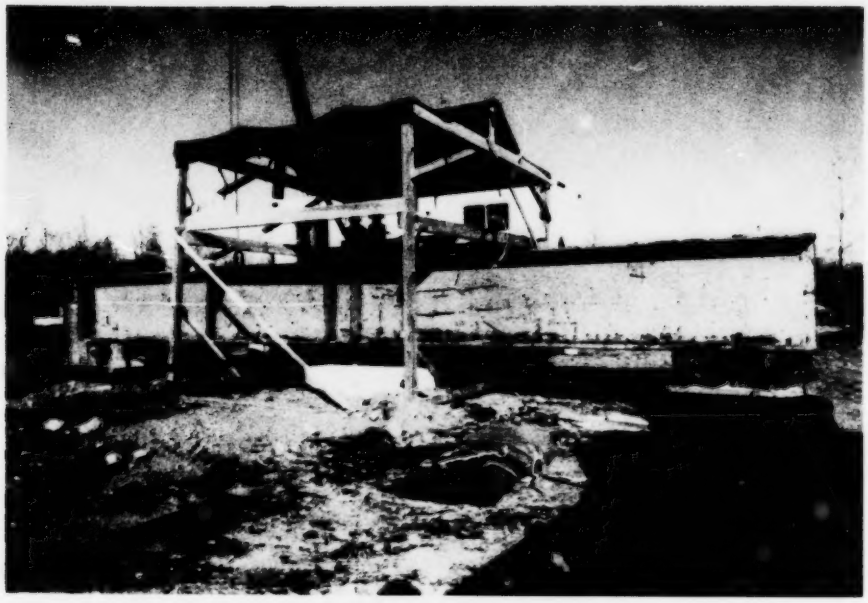

FIGURE 17
LONG TERM UPLIFT TEST ARRANGEMENT

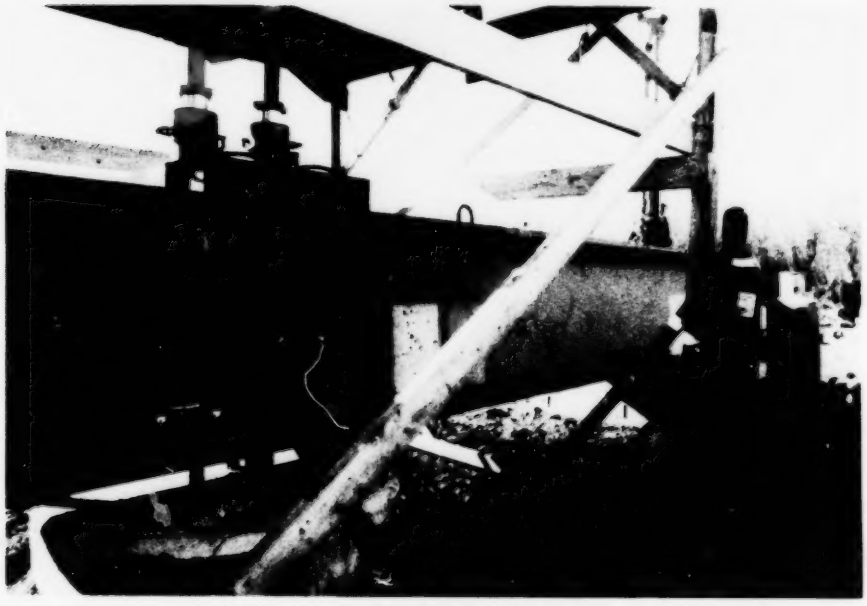

FIGURE 18
CLOSE - UP OF HYDRAULICS AND NITROGEN TANK
FOR THE LONG TERM UPLIFT TEST

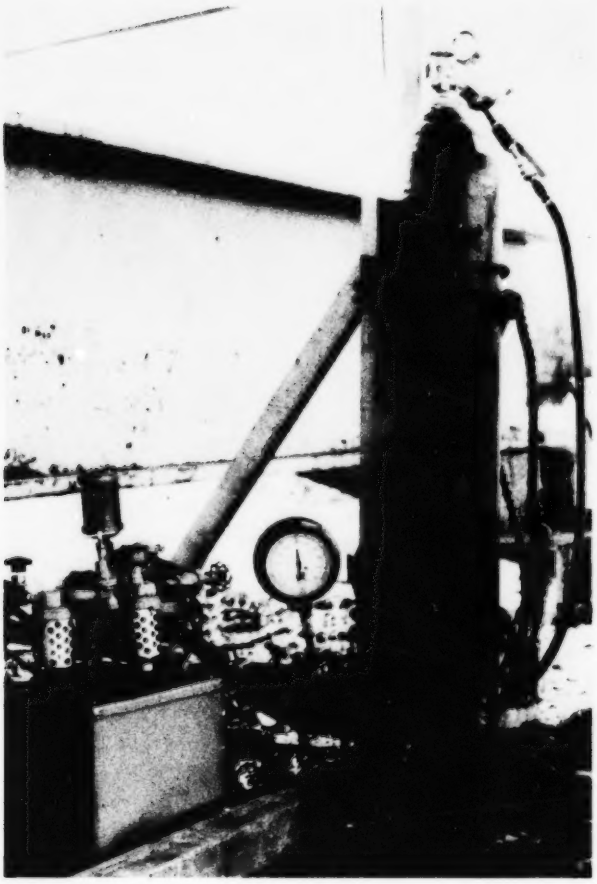

FIGURE 19

SPRAGUE PUMP AND NITROGEN TANK FOR MAINTAINING THE HYDRAULIC PRESSURE FOR THE LONG TERM UPLIFT TEST

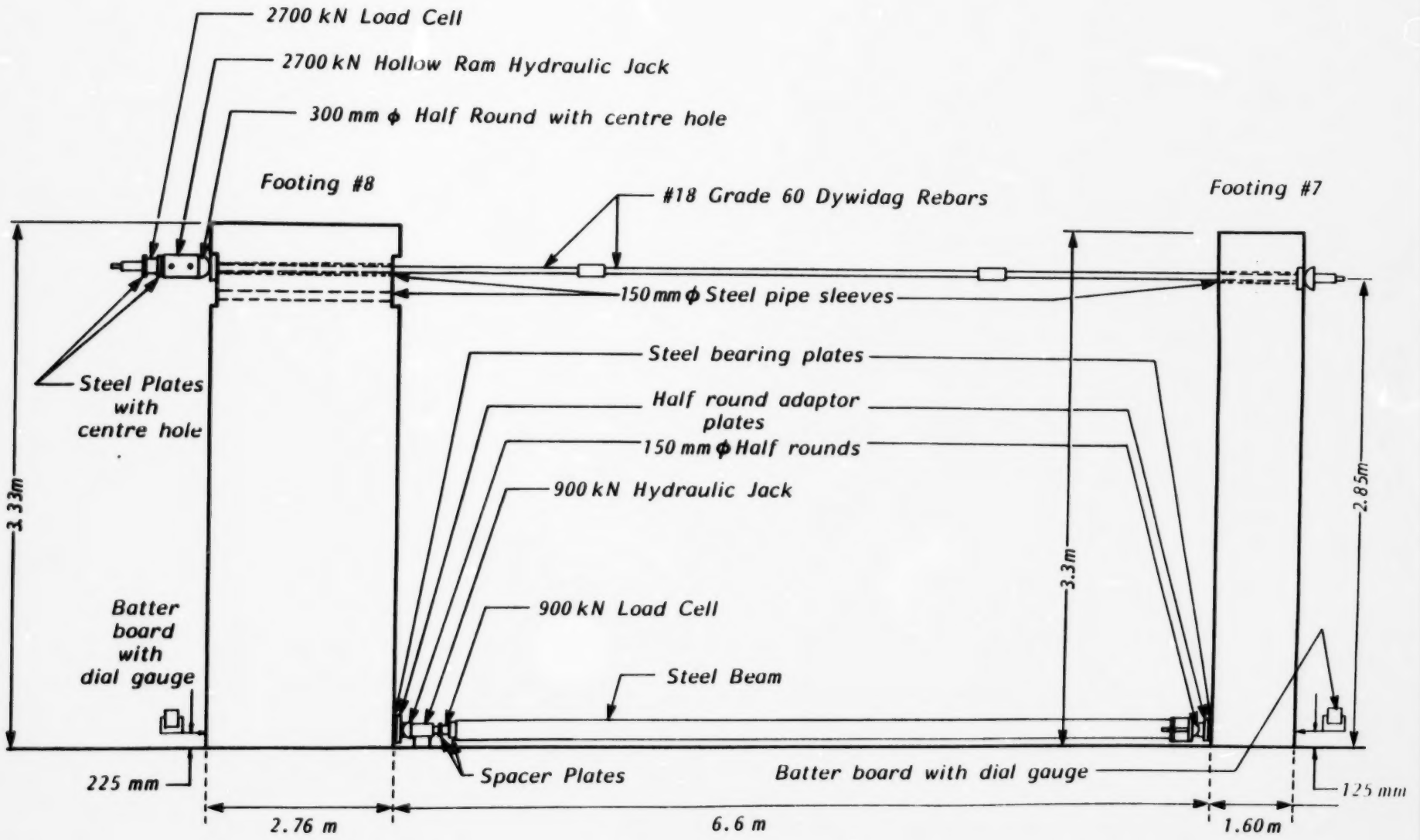

FIGURE 20

OVERTURNING MOMENT TEST ARRANGEMENT
(TEST NO. 10)

86-165 170527 R

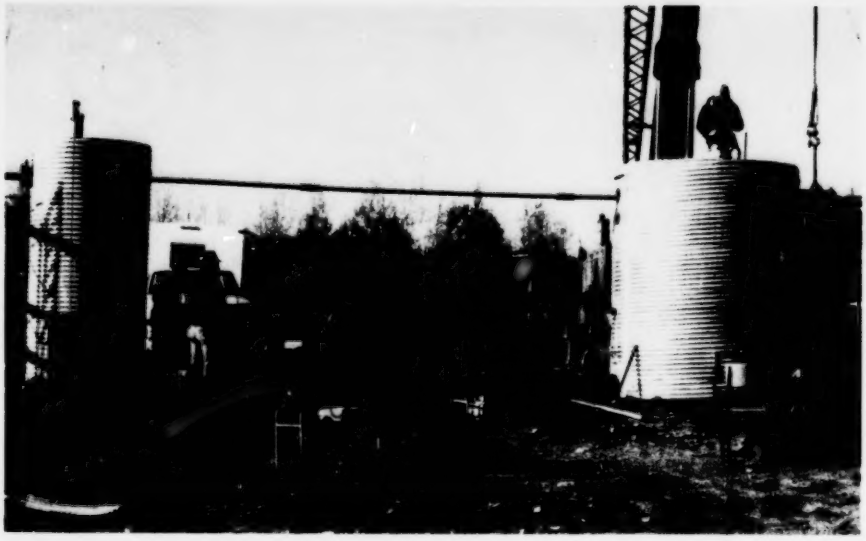

FIGURE 21

OVERTURNING MOMENT TEST (TEST NO. 10)

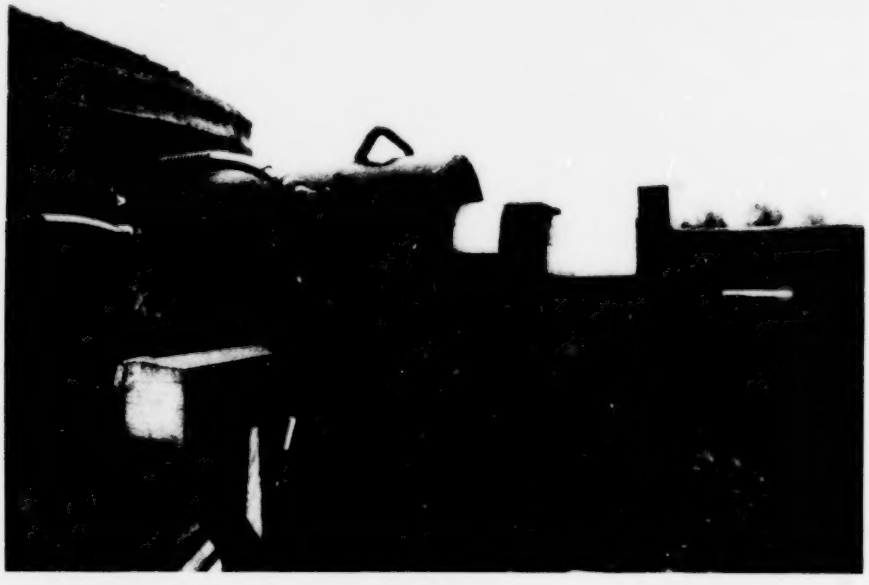

FIGURE 22

TOP LOADING ARRANGEMENT SHOWING HYDRAULIC JACK AND LOAD CELL

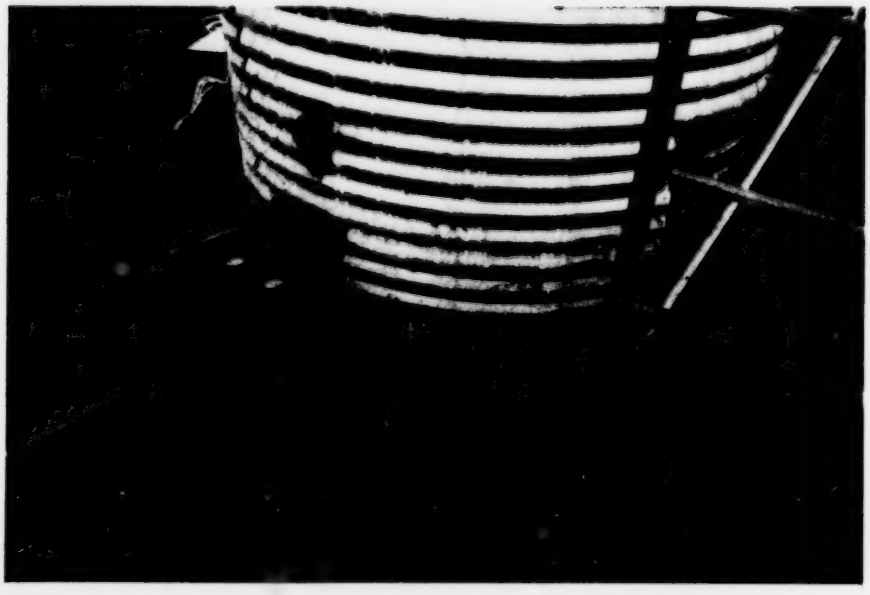

FIGURE 23

INSTRUMENTATION MONITORING GROUND LINE DISPLACEMENT OF FOOTING #7

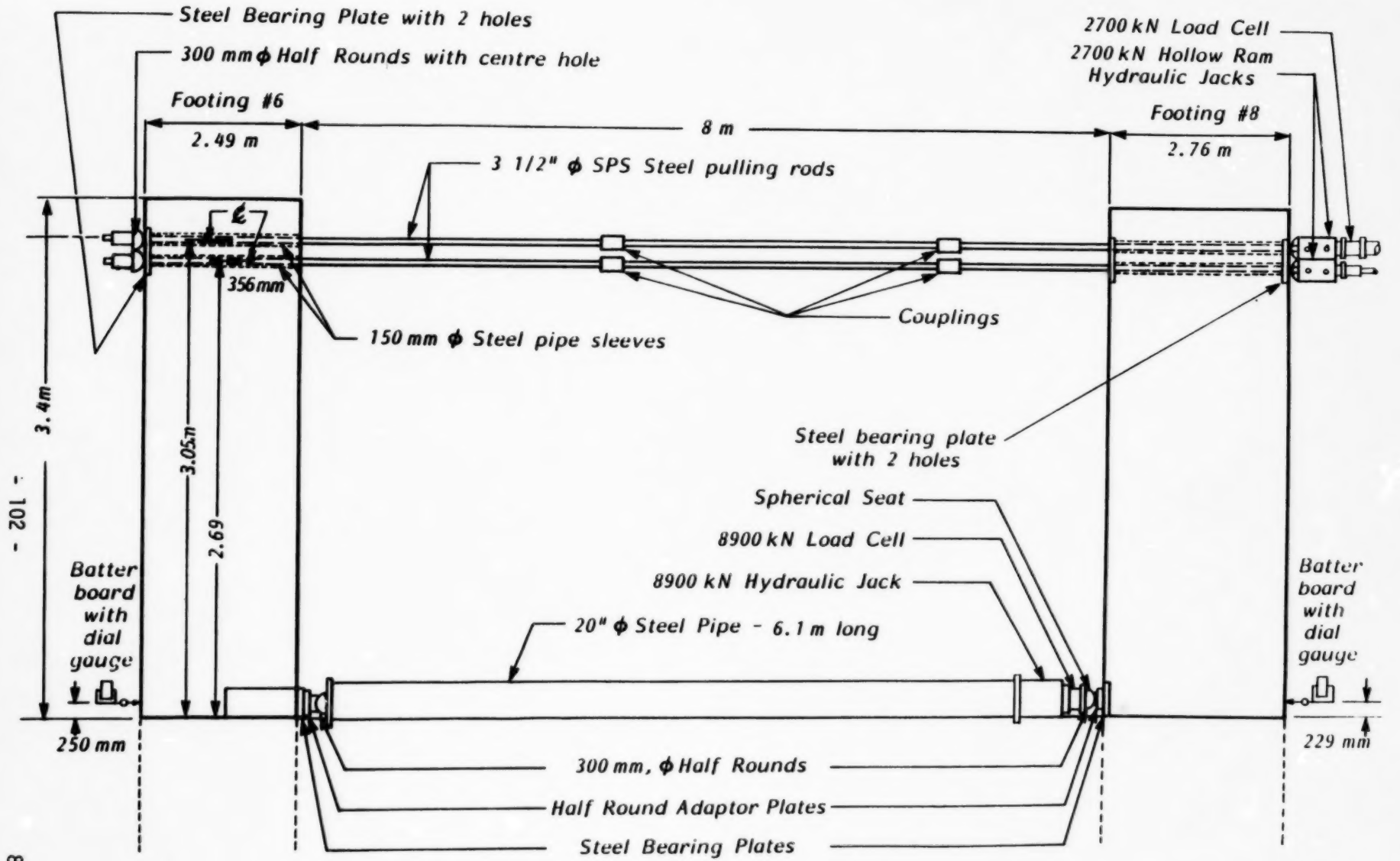

FIGURE 24

OVERTURNING MOMENT TEST ARRANGEMENT (TEST NO. 11)

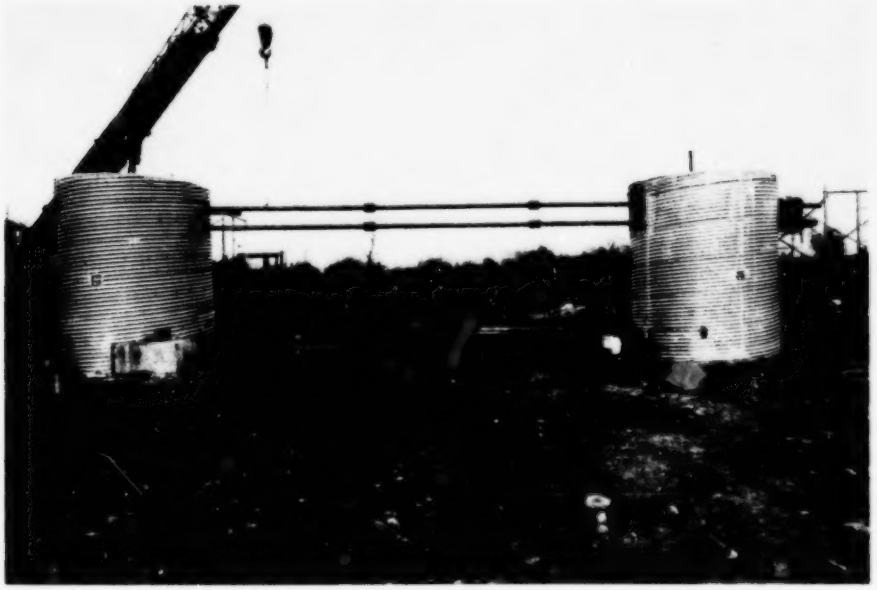

FIGURE 25

OVERTURNING MOMENT TEST ARRANGEMENT
TEST NO.11

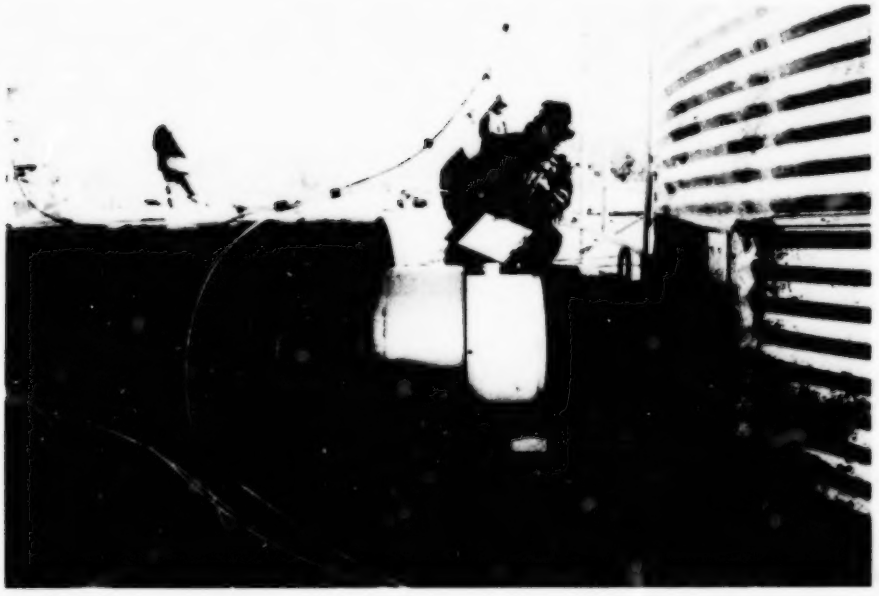

FIGURE 26

CLOSE-UP OF THE LATERAL LOAD
APPARATUS AT THE GROUND ELEVATION

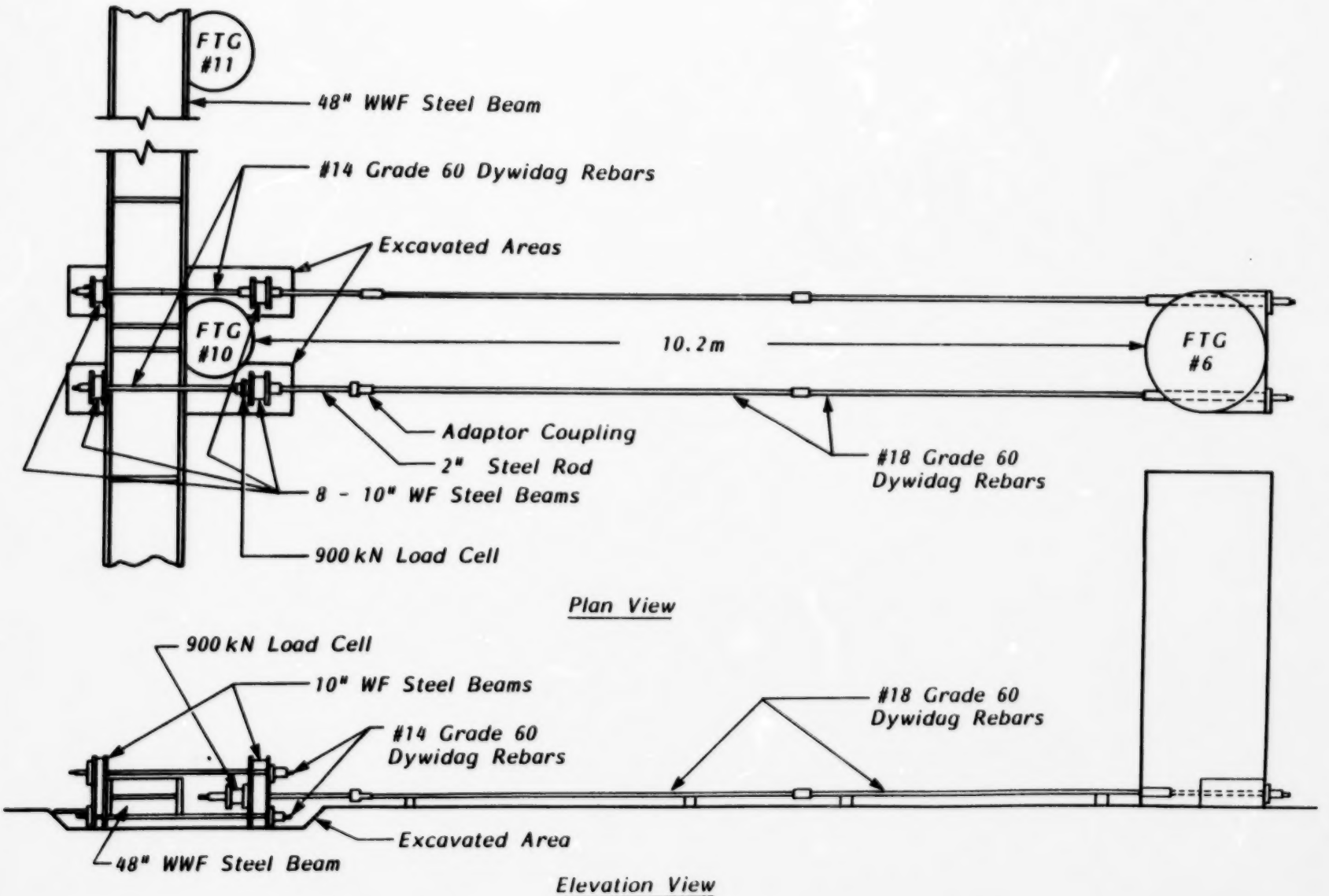

FIGURE 27

TIE-BACK ARRANGEMENT FOR TEST NO. 12

FIGURE 28

OVERTURNING MOMENT TEST NO. 12 WITH LATERAL TIE-BACK FROM FOOTING #6 TO FOOTINGS #9,10 & 11

FIGURE 29

TEST ARRANGEMENT FOR TIE-BACK TO FOOTING #6

FIGURE 30

ARRANGEMENT OF 10" WWF BEAMS, DYWIDAG REBARS AND 48" WWF TIE-BACK BEAM BEHIND 1.6 m FOOTINGS #9,10 &11

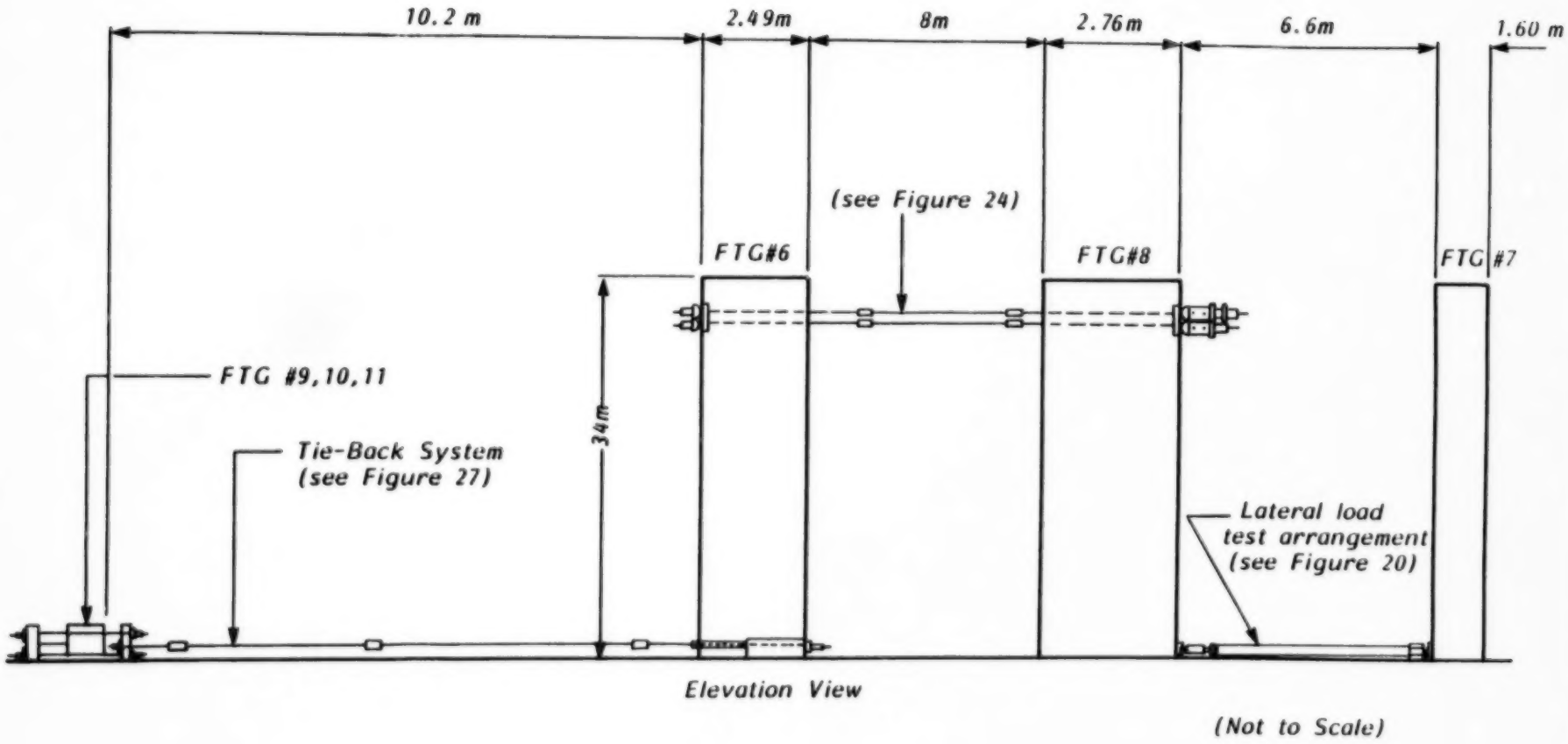

FIGURE 31

OVERTURNING MOMENT TEST ARRANGEMENT (TEST NO. 12)

86-165
170533 RD

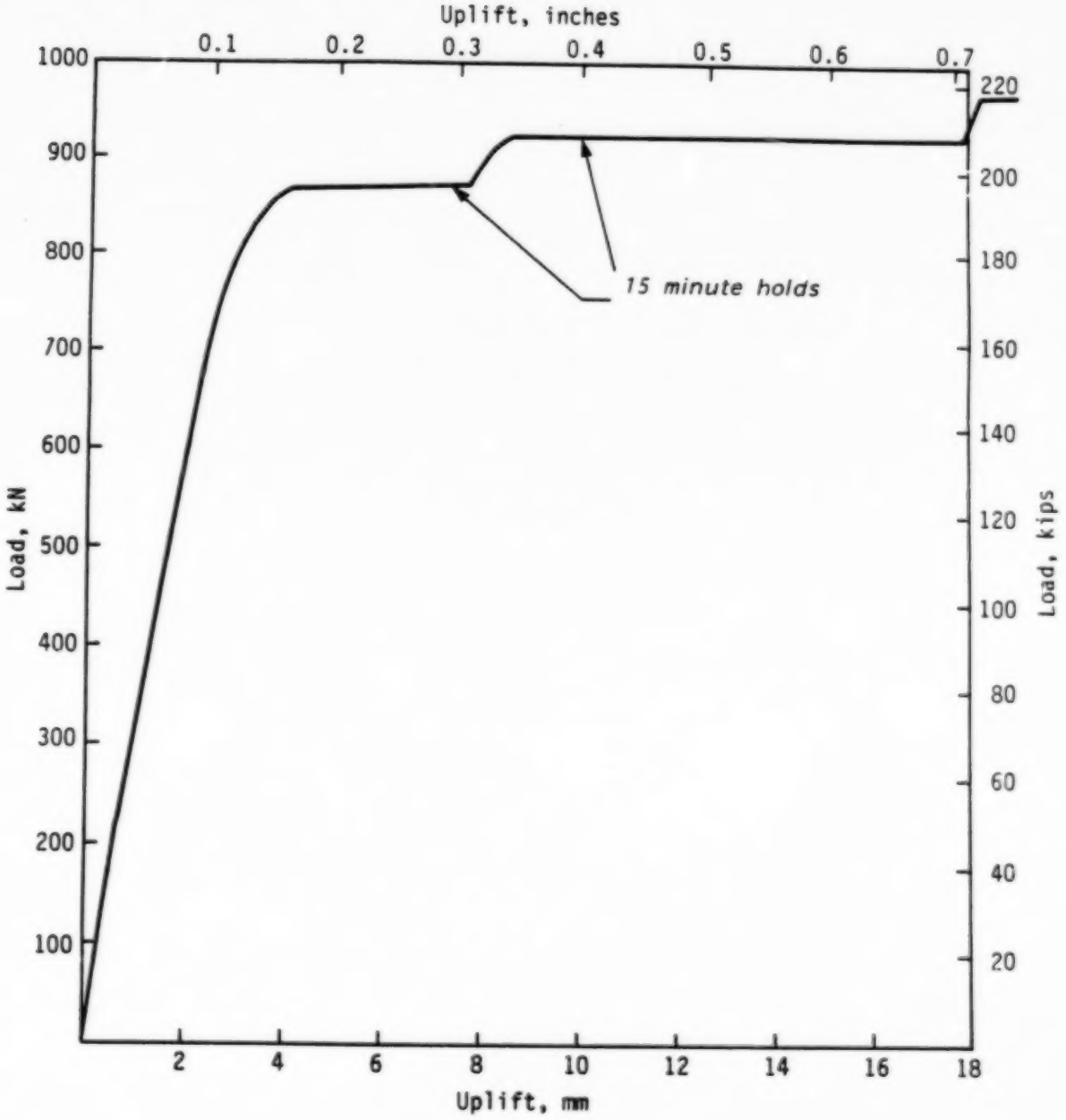

FIGURE 32
UPLIFT TEST ON FOOTING #9

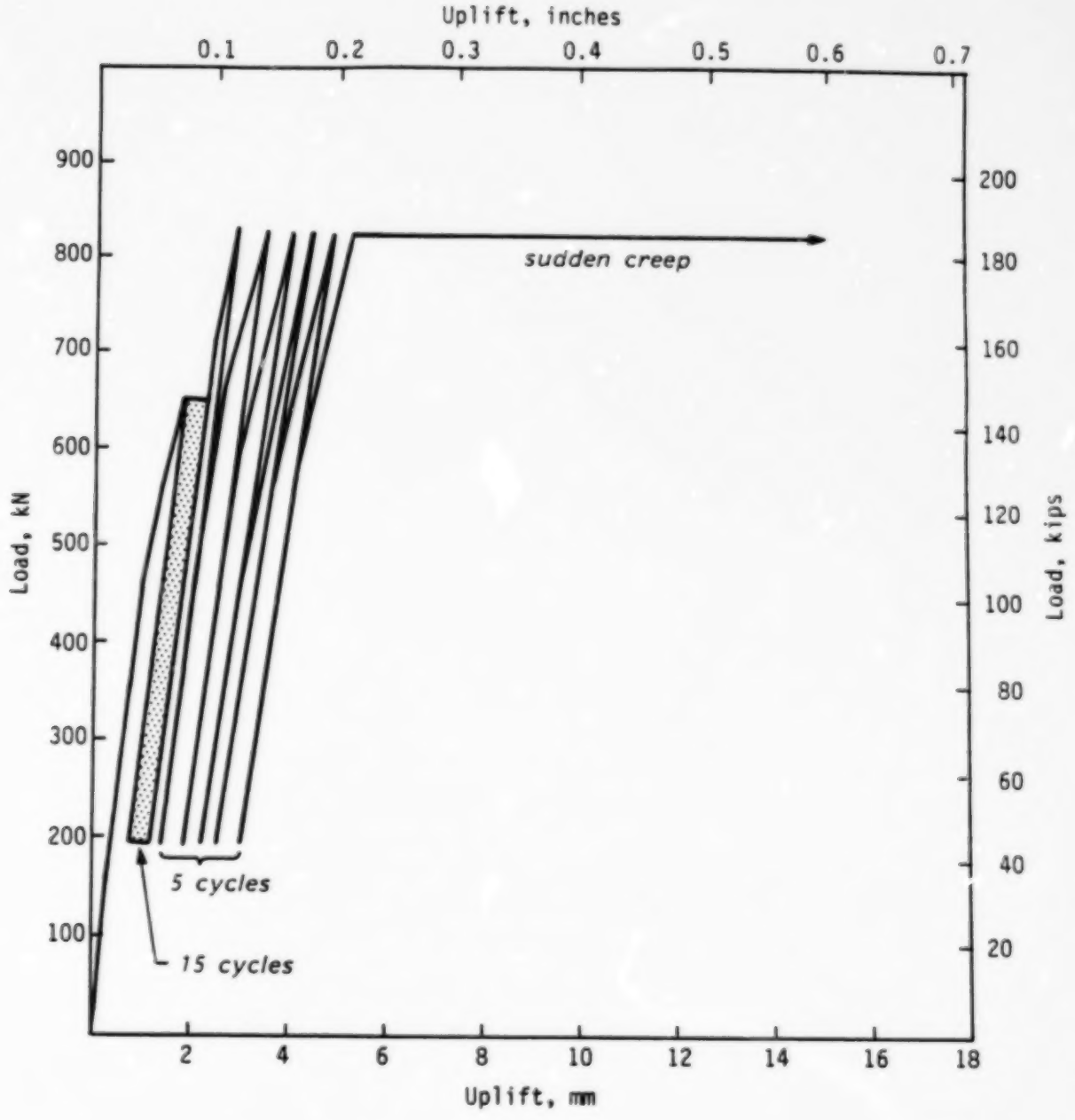

FIGURE 33
CYCLIC UPLIFT TEST ON FOOTING #11

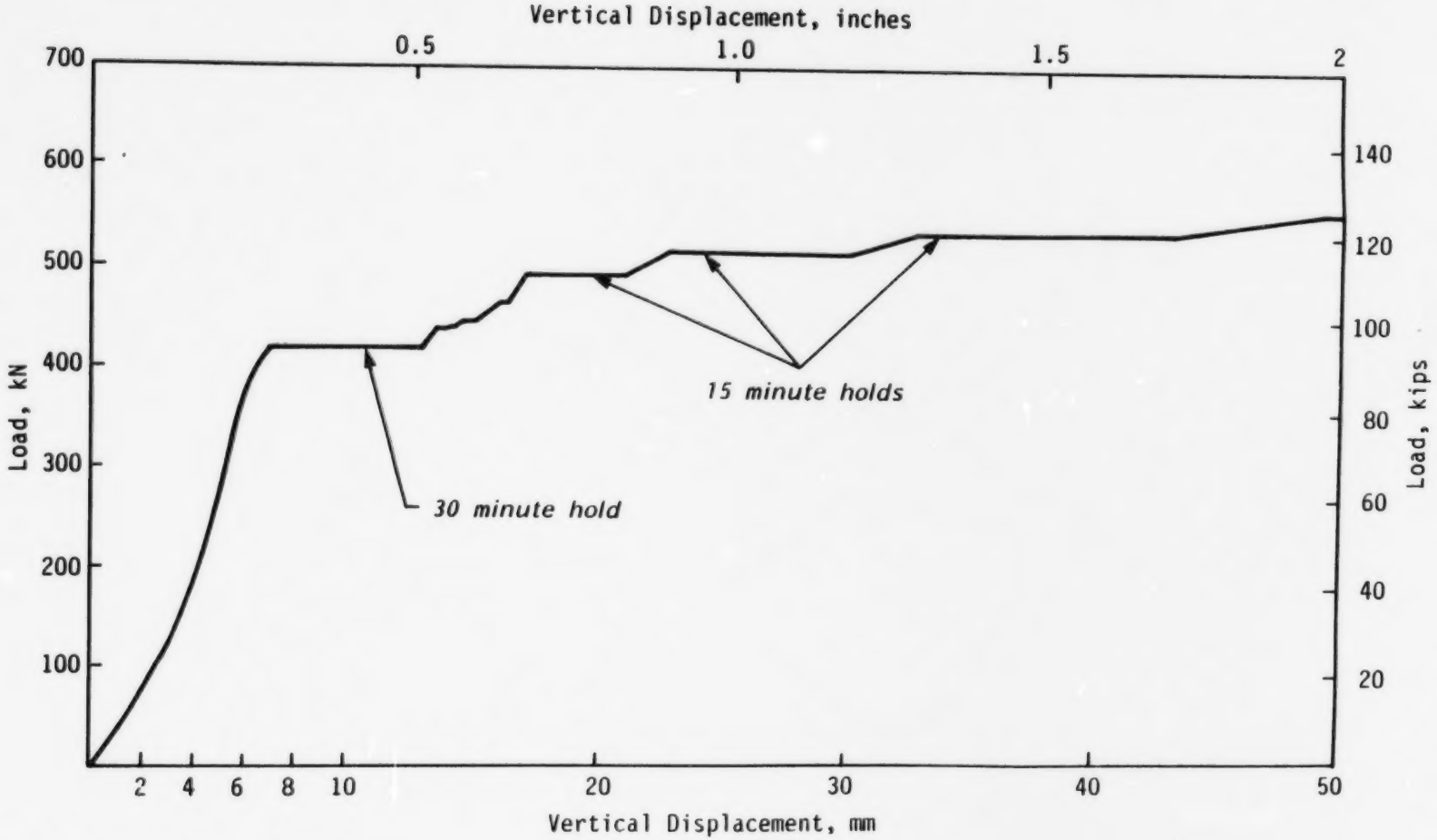

FIGURE 34
COMPRESSION TEST ON FOOTING #2

86-165
170536 RE

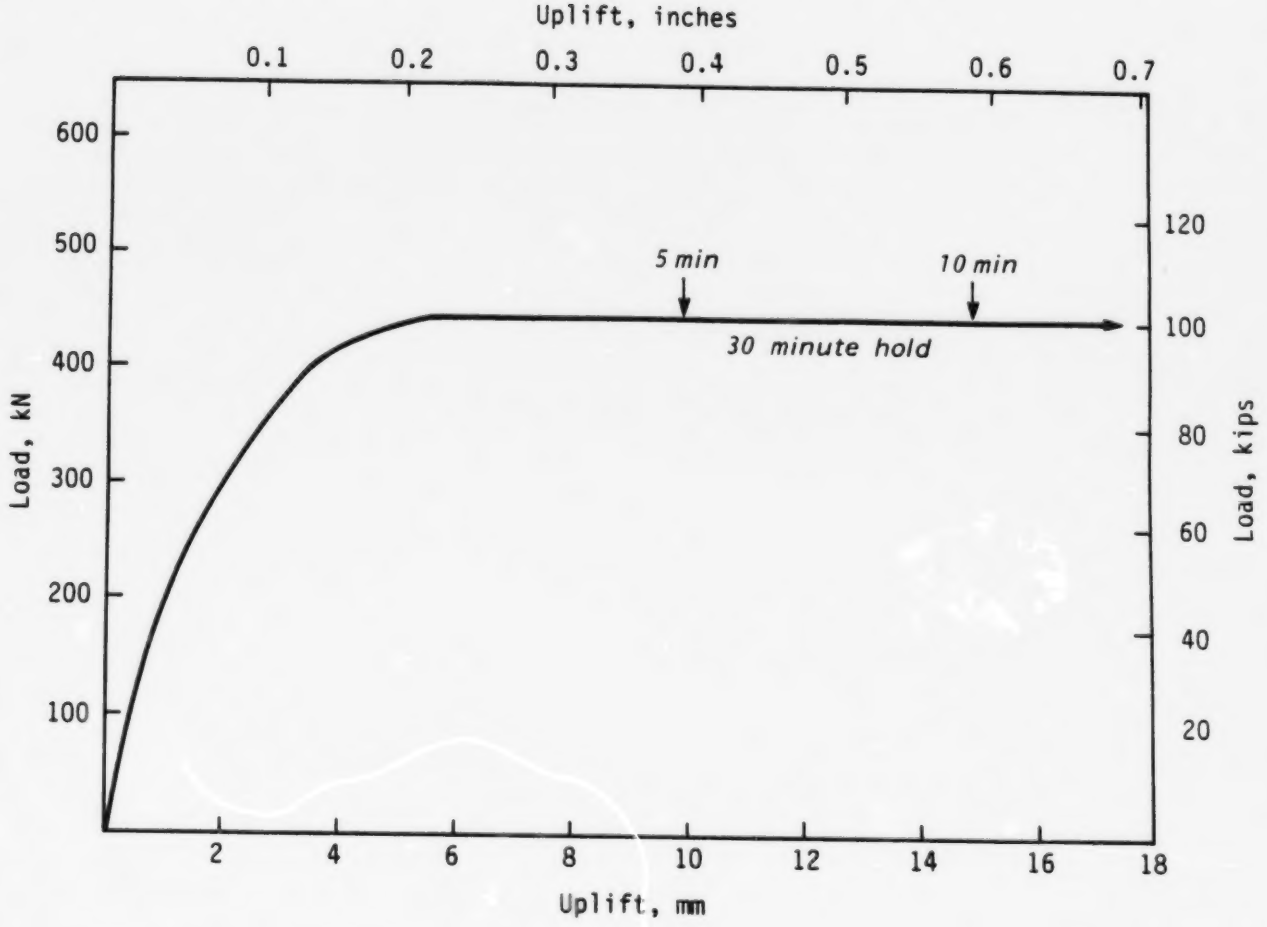

FIGURE 35
UPLIFT TEST ON FOOTING #1

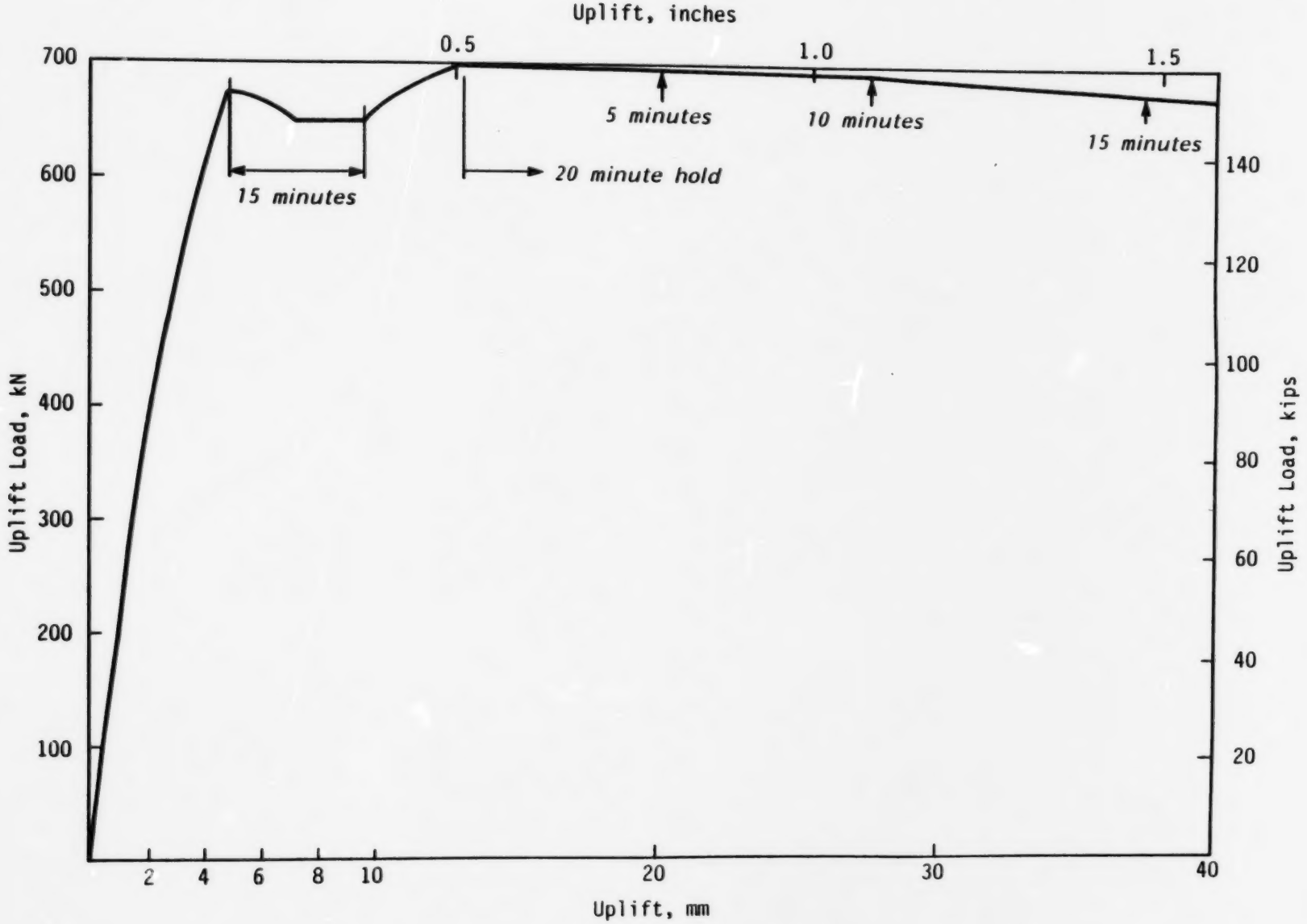

FIGURE 36
COMBINED LATERAL AND UPLIFT LOAD TEST ON FOOTING #3

86-165
170538 RD

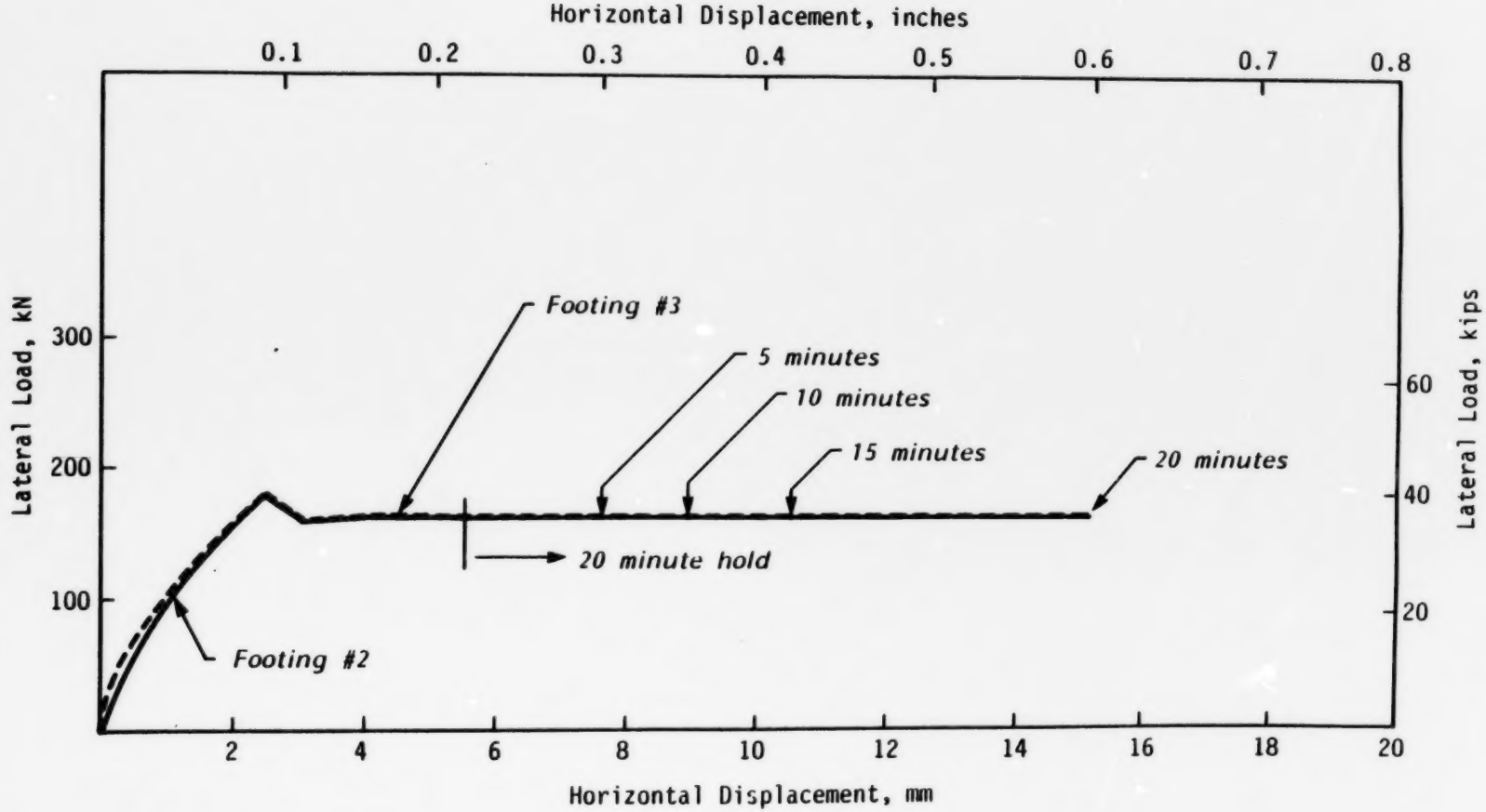

FIGURE 37
COMBINED LATERAL AND UPLIFT LOAD TEST ON FOOTING #3

86-165
170539 RD

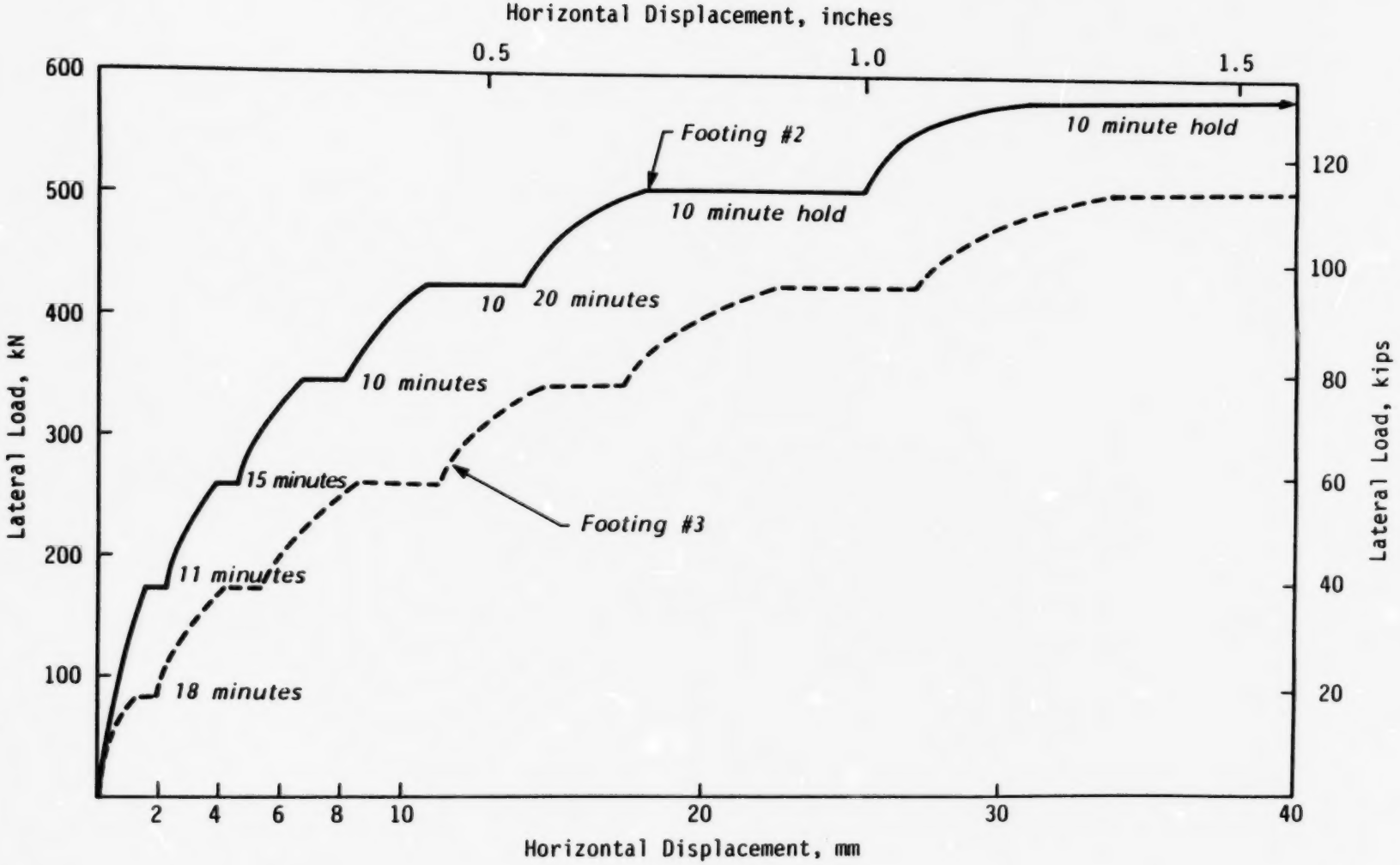

FIGURE 38

LATERAL LOAD TEST BETWEEN FOOTING #2 AND #3

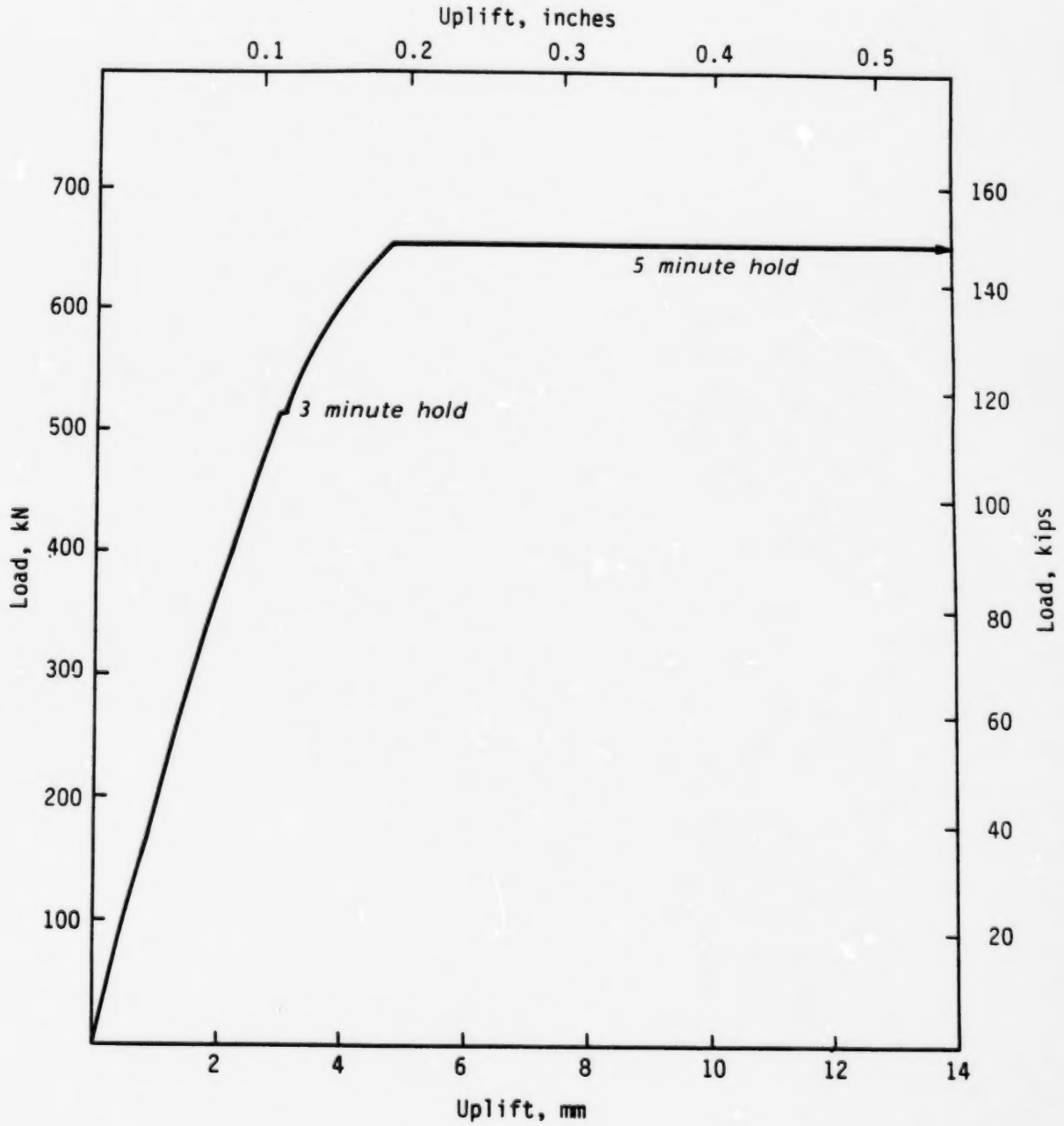

FIGURE 39
UPLIFT TEST ON FOOTING #10

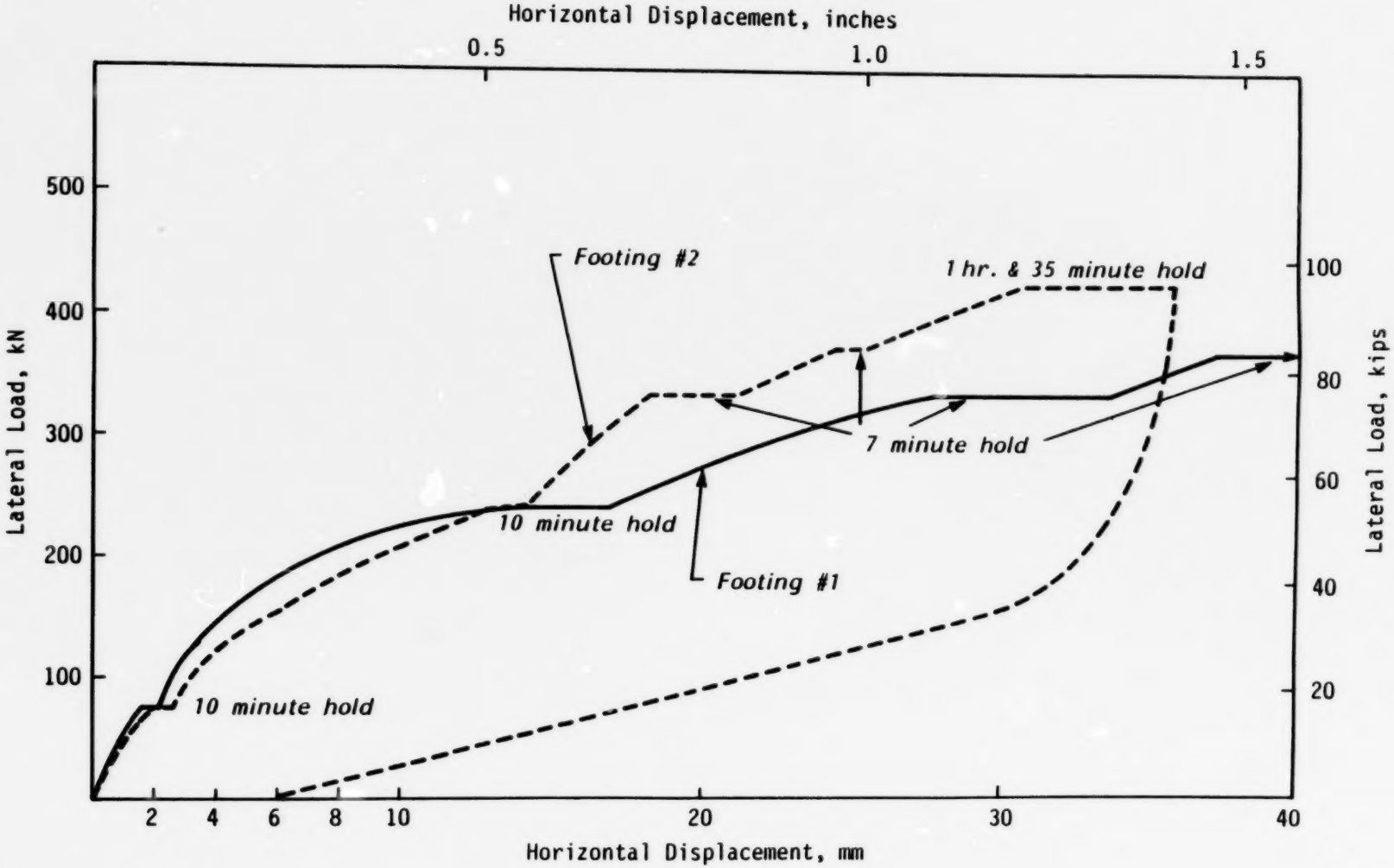

FIGURE 40

LATERAL LOAD TEST BETWEEN FOOTING #1 AND #2

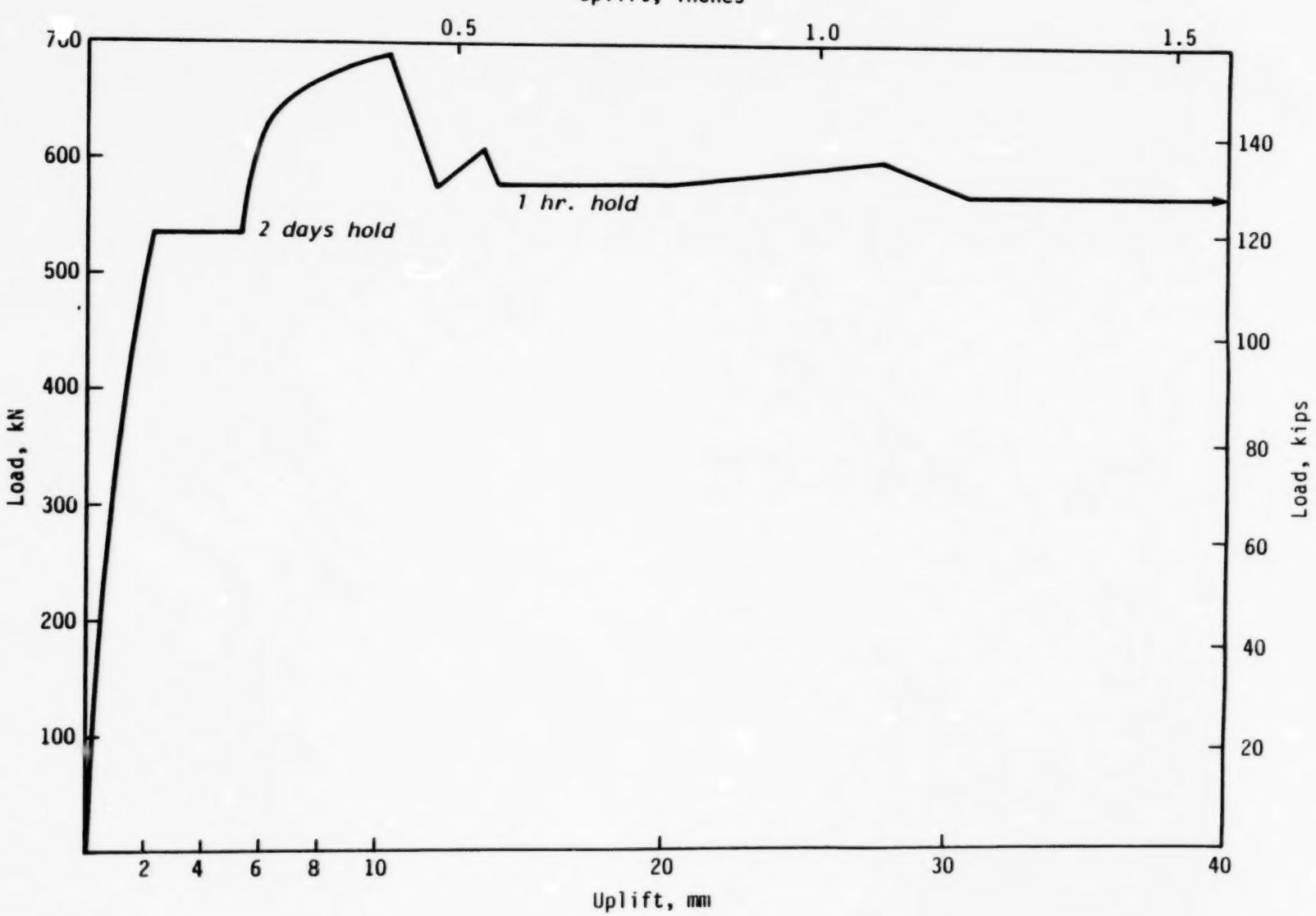

FIGURE 41

LONG TERM UPLIFT TEST ON FOOTING #4

86-165

170543 RD

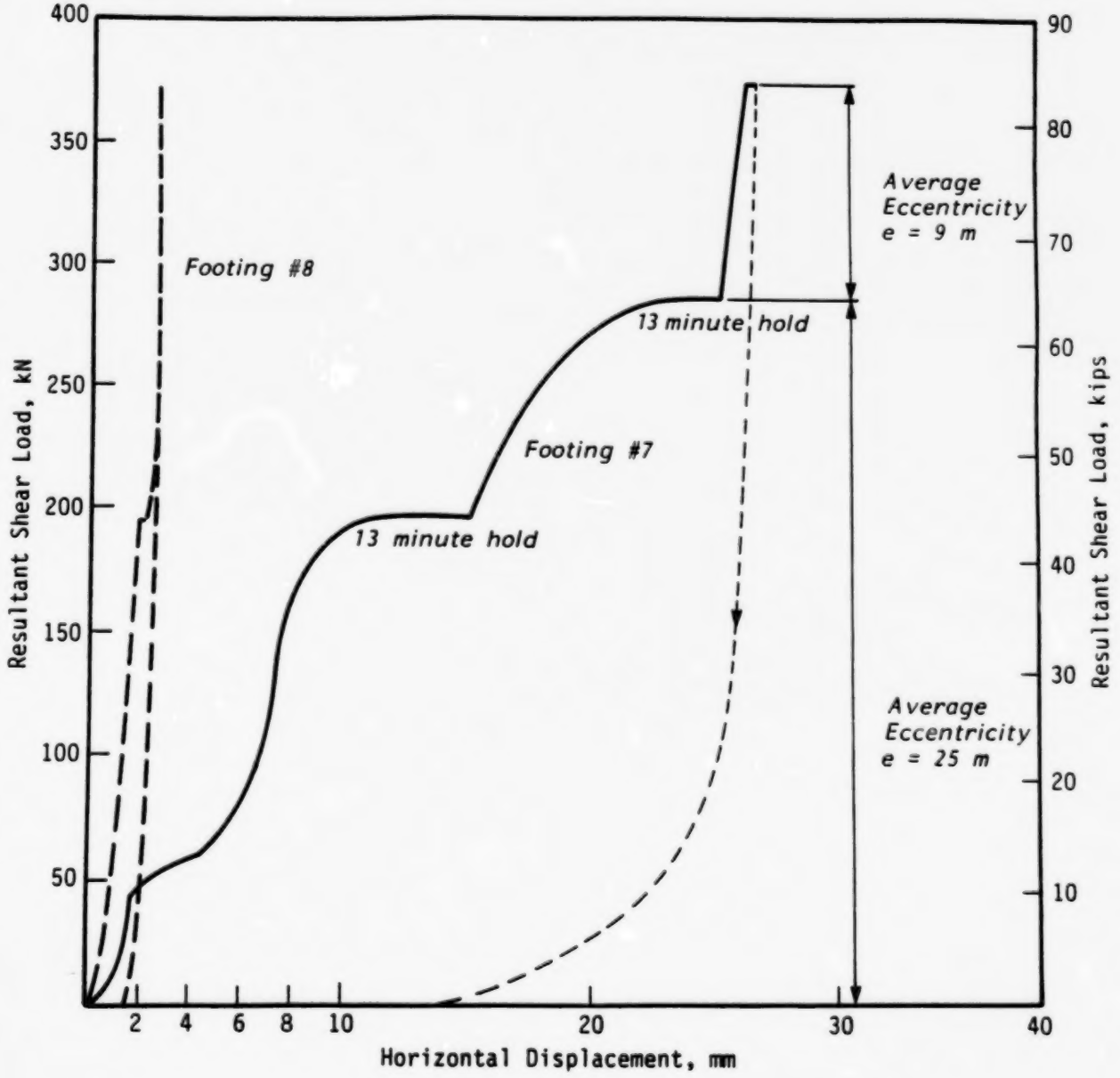

FIGURE 42

OVERTURNING MOMENT TEST BETWEEN FOOTING #7 AND #8 (WITH TOP & BOTTOM JACKS)

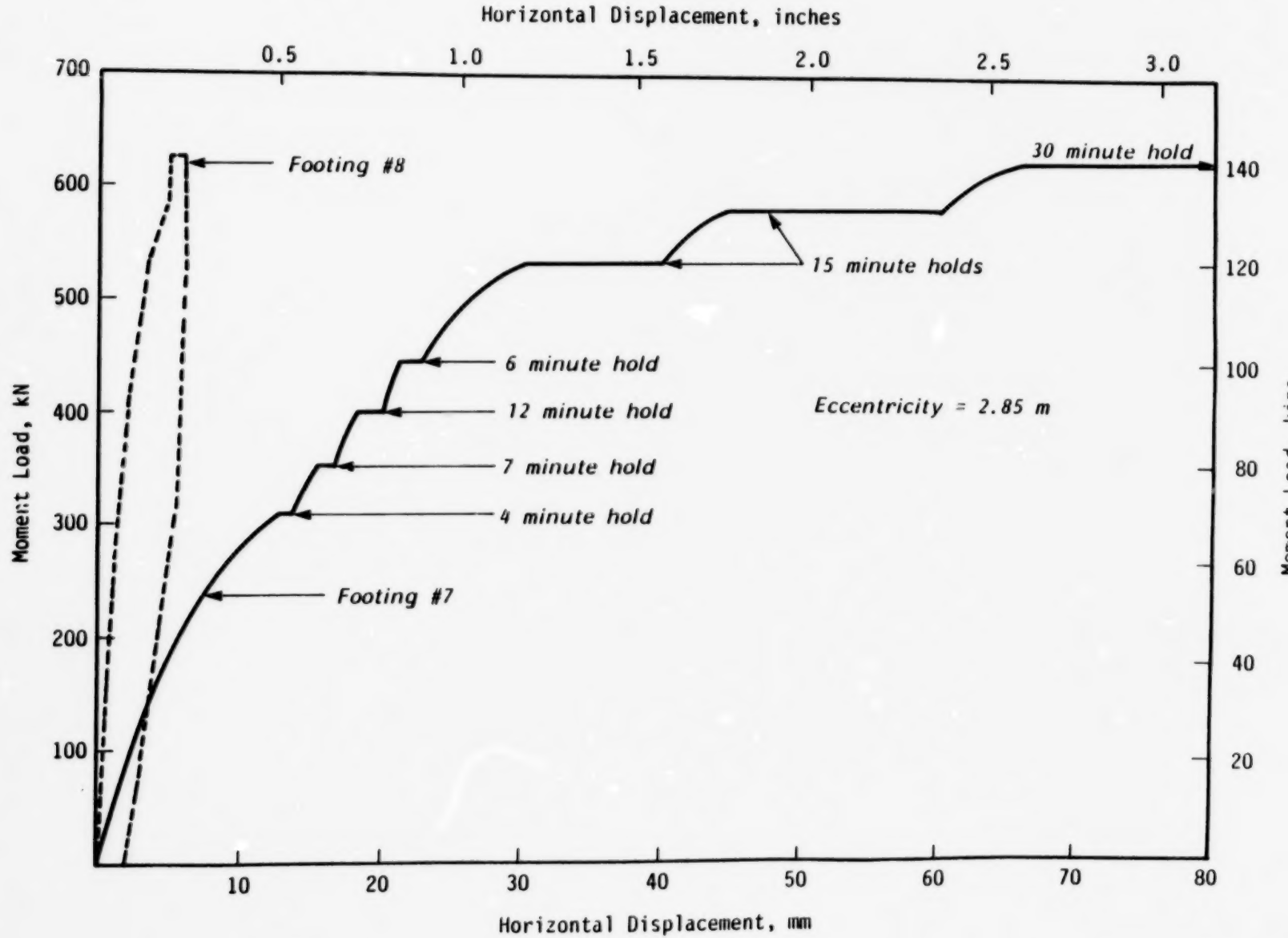

FIGURE 43

OVERTURNING MOMENT TEST BETWEEN FOOTING #7 AND #8 (WITHOUT BOTTOM JACK)

86-165 170545 RC

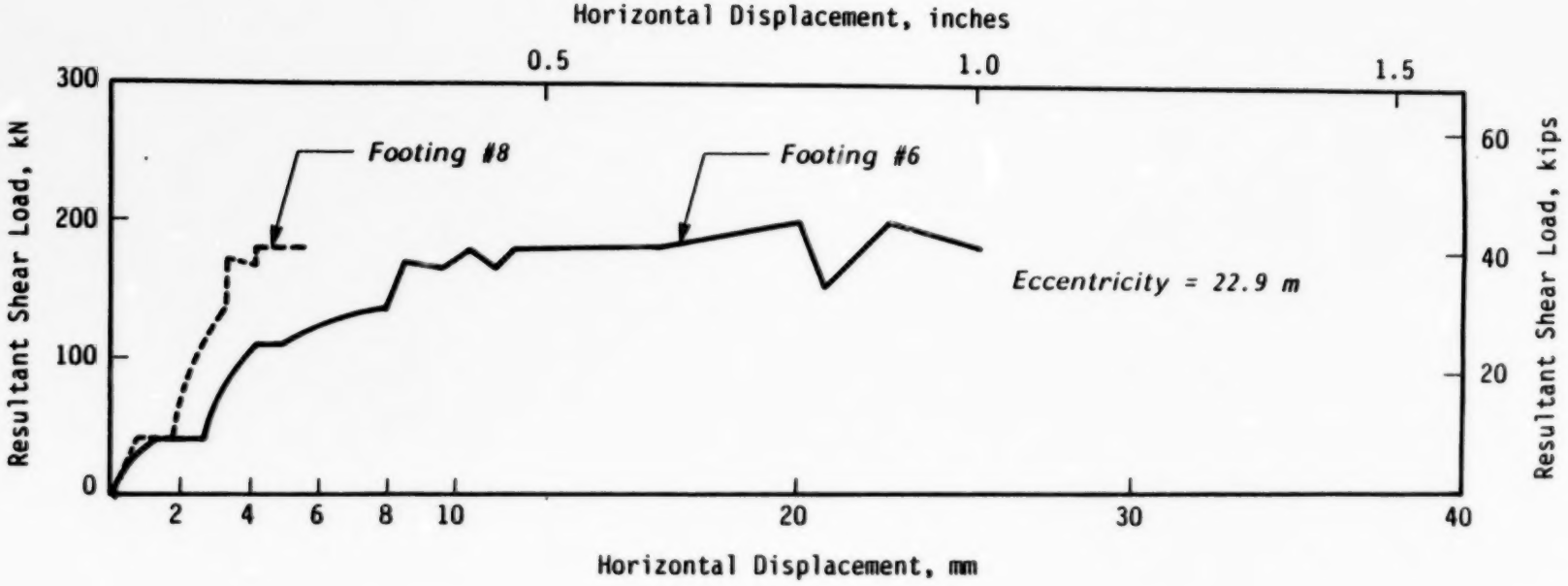

FIGURE 44

OVERTURNING MOMENT TEST BETWEEN FOOTINGS #6 AND #8
(WITH TOP AND BOTTOM JACKS)

86-165

170546 RD

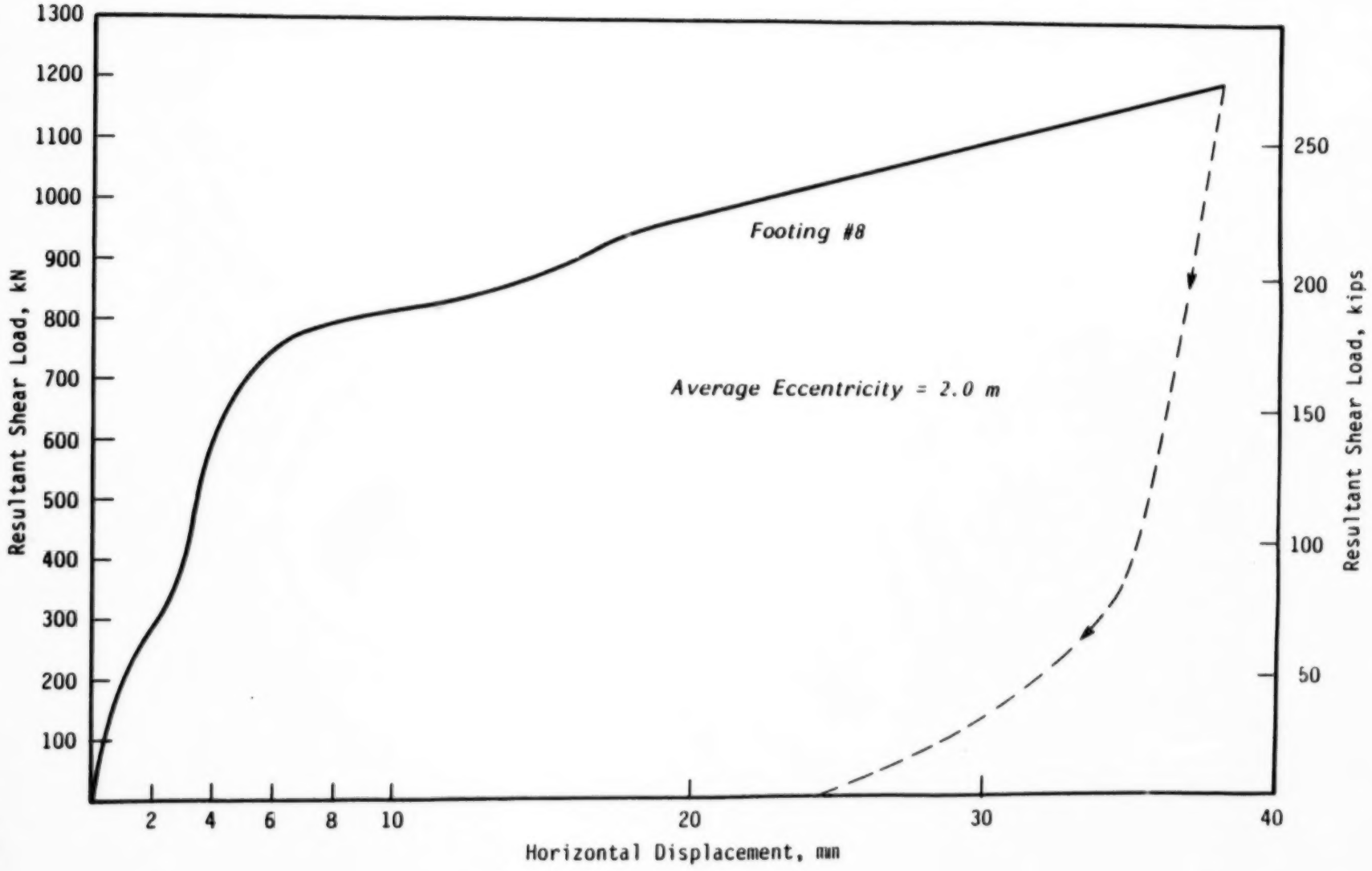

FIGURE 45

OVERTURNING MOMENT TEST BETWEEN FOOTINGS #6 AND #8
(WITH NO BOTTOM JACK)

86-165
170647 RD

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APPENDIX D

INSTRUMENTATION MONITORING RESULTS

TABLE 1D

Pressure Cell Readings During Compression Test

Applied Load on Footing 2		Cell Pressures at Base of Footing (kPa)				End Bearing Load	
kips	kN	Cell 1	Cell 2	Cell 3	Average	kips	kN
0	0	0	0	0	0	0	0
40	178	21	22	20	21	78	34
74	329	50	55	44	50	18.5	82
94	418	115	116	118	116	42.9	191
100	445	125	124	116	122	45.2	201
104	463	132	132	120	128	47.3	210
110	490	140	134	126	133	49.2	219
110*	490*	144	138	133	138	51.0	227
116	516	154	149	141	148	54.7	244
116*	516*	158	149	151	153	56.5	252
122	543	167	149	157	158	58.4	260
122*	543*	175	166	167	169	62.5	278
125	556	179	160	171	170	62.9	280
134	596	216	214	196	209	77.3	344

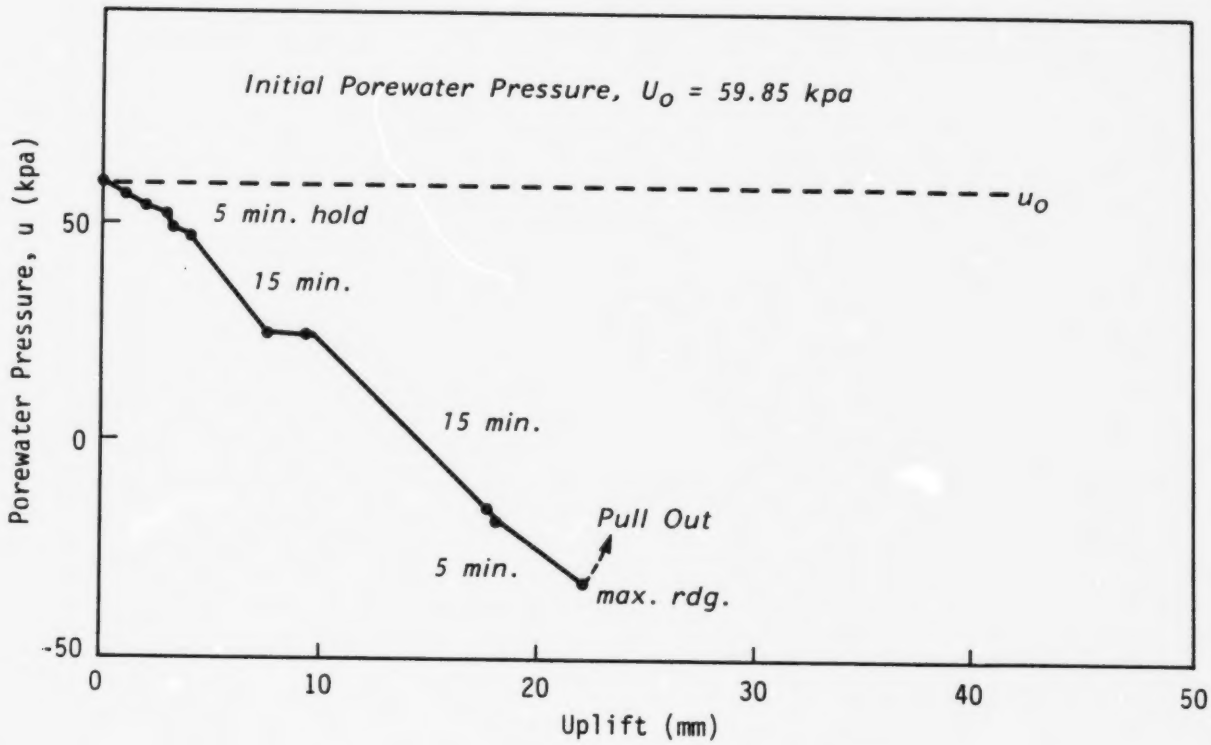

FIGURE 1D

POREWATER PRESSURE FOR UPLIFT TEST ON FOOTING #9

Test No. 1
Ground Elevation 29.80m
Elevation of Piezometer 22.73m
Elevation of Footing Base 22.84m

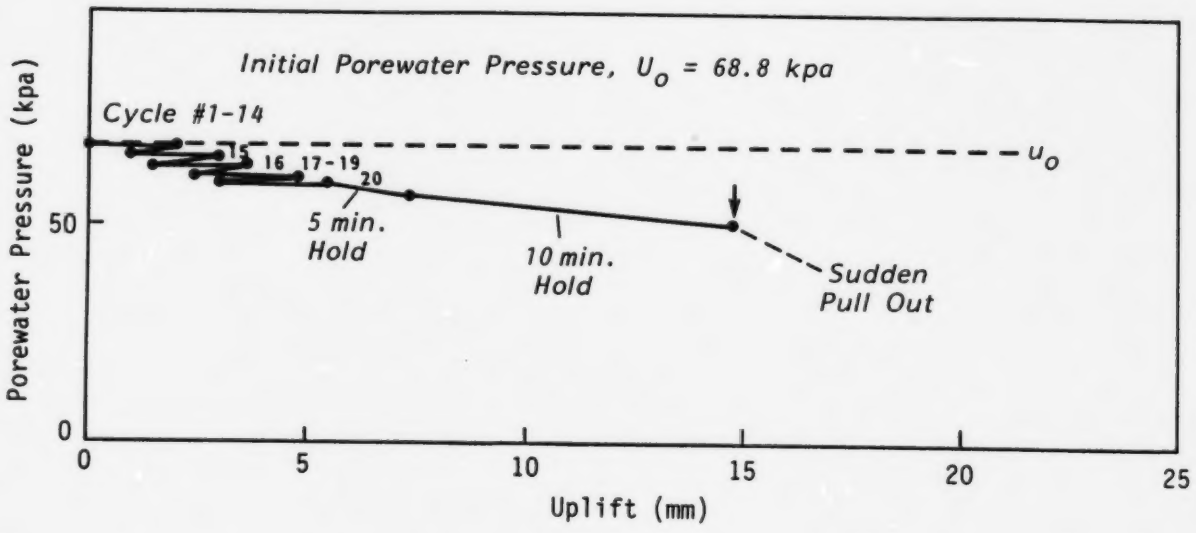

FIGURE 2D

POREWATER PRESSURE FOR CYCLIC UPLIFT TEST ON FOOTING #11

Test No. 2
Ground Elevation 29.60m
Elevation of Piezometer 22.58m
Elevation of Footing Base 22.87m

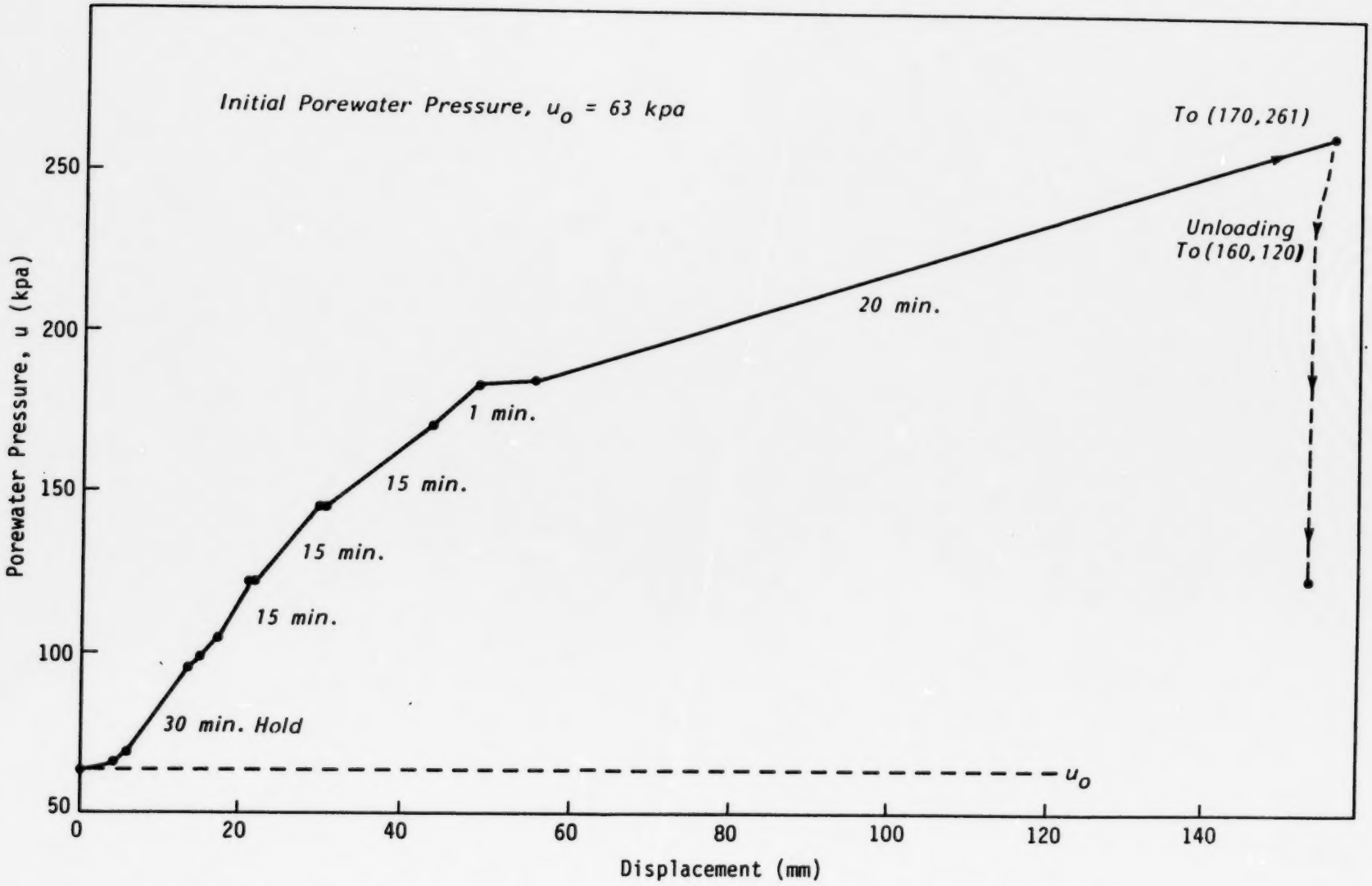

FIGURE 3D

POREWATER PRESSURE FOR COMPRESSION TEST ON FOOTING #2

Test No. 3
Ground Elevation 29.50m
Elevation of Piezometer 22.39m
Elevation of Footing Base 23.02m

171457 RD

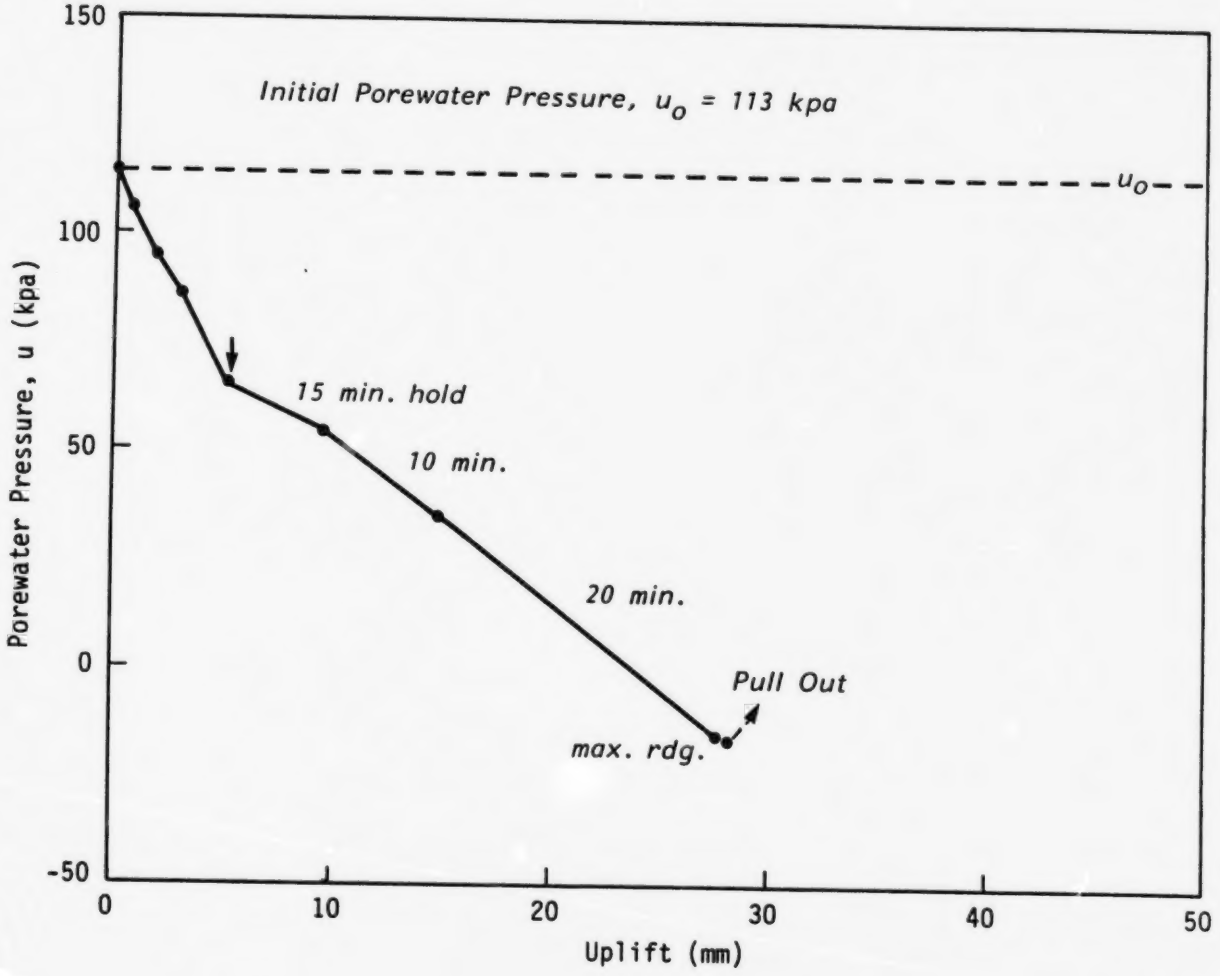

FIGURE 4D

POREWATER PRESSURE FOR UPLIFT TEST ON FOOTING #1

Test No. 4
Ground Elevation 29.50m
Elevation of Piezometer 18.91m
Elevation of Footing Base 19.77m

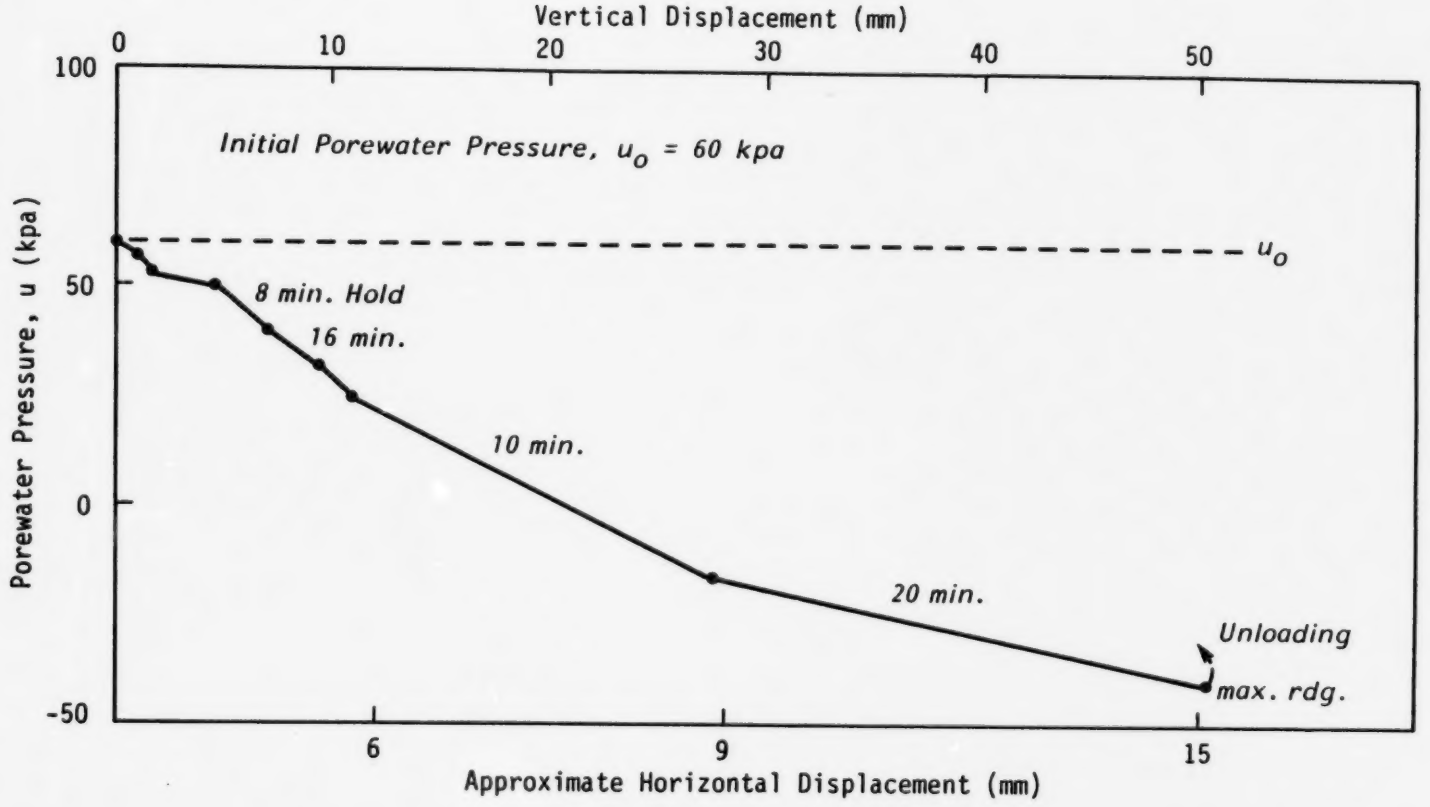

FIGURE 5D

POREWATER PRESSURE FOR COMBINED UPLIFT AND LATERAL LOADING ON FOOTING #3

Test No. 5
Ground Elevation 29.30m
Elevation of Piezometer 22.08m
Elevation of Foundation Base 22.82m

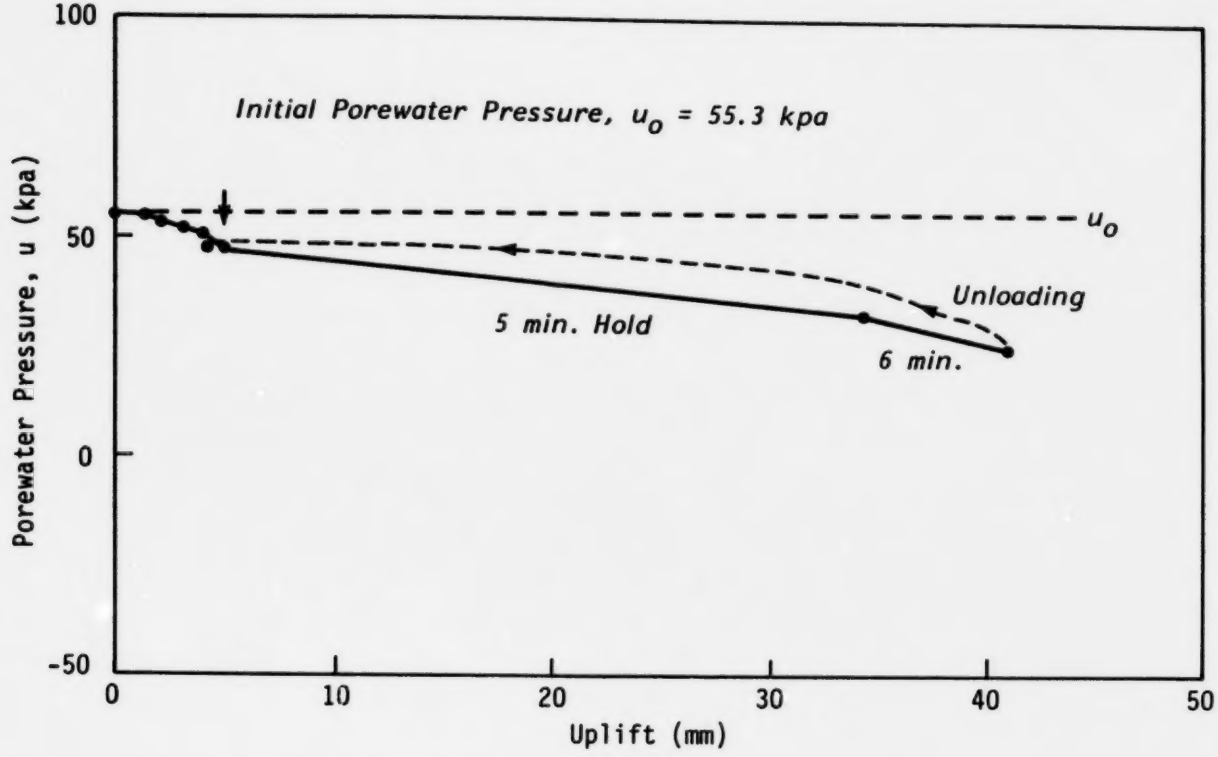

FIGURE 6D

POREWATER PRESSURE FOR UPLIFT TEST ON FOOTING #10

Test No. 7
Ground Elevation 29.80m
Elevation of Piezometer 22.71m
Elevation of Footing Base 23.07m

86-165

171458 RD

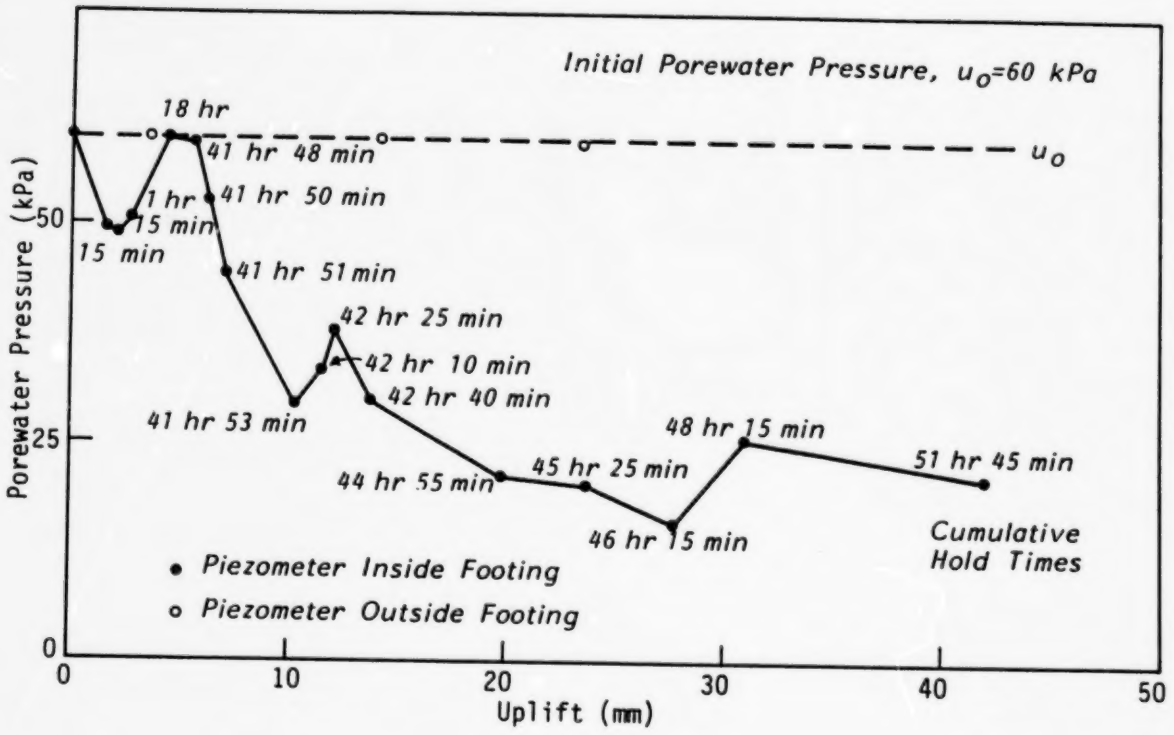

FIGURE 7D

POREWATER PRESSURE FOR LONG TERM UPLIFT ON FOOTING #4

Test No. 9
Ground Elevation 29.50m
Elevation of Piezometer Inside Footing 22.49m
Elevation of Piezometer Outside Footing 26.10m (at 15cm from edge of footing)
Elevation of Footing Base 22.69m